WASTEWATER MANAGEMENT STUDY
FOR

AD036643

LAKE MICHIGAN

CHICAGO-
SOUTH END of
LAKE MICHIGAN.

ENVIRONMENT

RESOURCES

PUBLIC PREFERENCE

TECHNOLOGIES

GOVERNMENT

APPENDIX B.

BASIS OF DESIGN

AND COST.

DEPARTMENT OF THE ARMY

CHICAGO DISTRICT CORPS OF ENGINEERS

219 SOUTH DEARBORN STREET
CHICAGO, ILLINOIS 60604

JULY 1973

REPORT COMPOSITION

The survey report is divided into a Summary, and 9 Appendices. A charge for each appendix and summary report to cover the cost of printing will be required, should purchase be desired. The appendices each contain a different category of information. Alphabetically identified, the appendices are:

A. Background Information - This appendix includes the population and industrial projections, wastewater flows and the engineering data used as a basis for planning.

B. Basis of Design and Cost - This appendix contains the criteria and rationale used to design and cost the final alternative wastewater treatment system components.

C. Plan Formulation - The appendix presents the planning concepts and procedures used in developing the alternative wastewater management plans that were examined during the study.

D. Description and Cost of Alternatives - This appendix contains a cost description and construction phasing analysis for each of the final five regional wastewater management alternatives. Components of these alternatives are described in detail in Appendix B.

E. Social - Environmental Evaluation - This report provides an assessment of the social and environmental impacts likely to arise from the implementation of the final five alternatives.

F. Institutional Considerations - This report presents an assessment of the institutional impacts likely to arise from implementation of the final five alternatives.

G. Valuation - This appendix presents a broad evaluation of the implications and use potential inherent in the final five alternatives.

H. Public Involvement/Participation Program - This appendix documents the program used to involve the public in the planning process.

I. Comments - This appendix contains all of the formal comments from local, State and Federal entities as the result of their review of the other appendices and the Summary Report. Also capsulized are the views of citizens presented at public meetings.

The Summary document presents an overview of the entire study.

WASTEWATER MANAGEMENT STUDY
CHICAGO-SOUTH END OF
LAKE MICHIGAN AREA

TECHNICAL APPENDIX B
BASIS OF DESIGN AND COST

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DEPARTMENT OF THE ARMY

Chicago District, Corps Of Engineers

219 South Dearborn Street

Chicago, Illinois 60604

WASTEWATER MANAGEMENT STUDY
CHICAGO-SOUTH END OF
LAKE MICHIGAN AREA

Prepared for the Chicago
District Corps of Engineers,
Department of the Army by
Bauer Engineering, Inc.

TECHNICAL APPENDIX B
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ACKNOWLEDGEMENT

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TECHNICAL APPENDIX B

INTRODUCTION

I. INTRODUCTION

A. ORIENTATION

This volume is a part of the United States Army, Chicago District, Corps of Engineers, Survey Scope Study Report for Regional Wastewater Management in the Chicago-South End of Lake Michigan area. The overall Survey Scope Study report consists of a Summary volume and supporting appendices. This appendix, Appendix B - Basis of Design and Cost, contains the basis of the design and costs for the five regional wastewater management systems presented in the Summary volume, and detailed in Appendix D, Description and Cost of Alternatives.

Attached to Appendix B is Data Annex B - Basis of Design and Cost, which presents more detailed, supporting information.

I. INTRODUCTION

B. DESCRIPTION OF REGIONAL WASTEWATER MANAGEMENT

CONCEPT OF REGIONAL MANAGEMENT

Water, in its perpetual movement through the hydrologic cycle, knows no political boundaries. This fact has a direct bearing on any consideration of alternative waste treatment management systems. In the Chicago area, wastewater flows from a large region and two states impact on the same water resource. Therefore, a management plan must include regional and interstate considerations.

Water has always been properly recognized as one of the necessities for life on this planet. It constitutes the major portion of our bodies, performs countless tasks that influence our personal lives, works in our plants and factories, transports our goods, and helps to grow our food. This extensive use exacts a heavy toll on the quality of water in the form of pollution. In its polluted form, water is no longer the life-giver and sustainer. In flowing through our streams and rivers it can kill; fish and other aquatic life are its first victims. The degradation of our environment is a simultaneous effect.

To restore the water to its usable state, we must institute controls over its cyclic flows through our environment; consequently, we must manage it. To manage it wisely, and with real insight into its nature and characteristics, we must manage it on its terms as well as ours. This then brings us back to the statement made in the beginning of this introduction. Water in its movement through our environment generally knows no other boundaries but those placed on it by nature itself. It follows the patterns of the hydrologic cycle, the laws of gravity, and flows within the natural drainage confines of the surface. Regional management systems recognize these natural boundaries as far more binding than those instituted on an area by a political institution. Other factors are also brought into play before final boundaries for a particular wastewater management unit are established by the persons charged with this responsibility. These factors include natural interrelation between the adjacent watersheds, demographic area development (present and future), economic efficiencies in management through economy of scale and investment planning, and management of all water resources including the renovated water.

Recent work undertaken by the U.S. Army Corps of Engineers, with the cooperation of the Federal Environmental Protection Agency, as well as the State and local water control agencies, is designed to bring the idea of regional management of wastewater into a sharper focus.

The regional wastewater management study for the Chicago and South End of Lake Michigan (C-SELM) area addresses itself to the idea of regional solutions of wastewater related problems. The region encompasses some 2,600 square miles of area, and includes watersheds of four major river systems. It is wedged between Lake Michigan on the east and two other major river basins. The Fox River basin lies to the west and the Kankakee River basin to the south of the C-SELM area. Politically, the C-SELM impacts on seven counties located within two states and also affects host of other agencies with political powers. Demographically, it impacts on a current population of seven million people and a projected population of eleven million by the year 2020. It envisions this population living in densely populated urban centers, suburban areas with lesser density, and loosely populated rural areas.

The planning time frame for this study covers a period of 50 years, with the final system design being based upon criteria applicable to the year 2020. Nineteen-ninety is used as the immediate planning date, by which the regional system would be initially implemented on a study area-wide basis.

WATER QUALITY GOALS

The goals of any wastewater renovation process can be expressed in terms of the anticipated resource uses. These uses include an aesthetic environment surrounding the watercourses, recreation opportunities created within the watercourse regime, healthy aquatic population within the stream regime and increased potable water supply. The renovating process, therefore, must have as its basis some specified water quality goals that will sustain these uses. The goals of water quality for the C-SELM study area were established with the aid of publications such as Report of the Committee on Water Quality Criteria prepared by the National Technical Advisory Committee to the Secretary of the Interior, (1968).

Water quality is virtually identical with effluent quality in C-SELM area. Groundwater-induced base flow comprises less than ten

percent of the daily average watercourse flow in those streams which do not recieve dilution flows from Lake Michigan. The other and predominant sources of C-SELM watercourse flow, municipal and industrial wastewater and stormwater surface runoff and infiltration, require treatment in order to meet water quality goals. Thus, virtually all C-SELM waters are comprised of effluents, and water quality becomes effluent quality.

Effluent quality goals to support the water quality goals were derived reflecting the finite performance capabilities of the best available technology, as communicated by technical publications and pilot demonstration programs. These effluent goals have come to be known as "No Discharge of Critical Pollutant" (NDCP) goals. In summary, the NDCP goals characterize a water quality that manifests all known concerns of water quality consistent with the capabilities of the specific type of treatment technology.

A second set of effluent goals and practices is also utilized in the C-SELM study for reference purposes. This latter set is comprised of the existing effluent standards and practices in Illinois and Indiana, respectively. For the presentation and detailed discussion of water quality goals and existing water quality standards the reader is referred to Data Annex B, Section I-B.

MANAGEMENT COMPONENTS

To satisfy the water quality goals outlined above, a wastewater management policy can be formulated. The policy deals with an array of management system components which, taken together, serve to intercept, treat, and return to the area waterways all water that has been polluted through use. This water may be municipal effluent, industrial effluent, or stormwater infiltration or surface runoff flows resulting from rainfall or snowmelt. The individual management system components are briefly introduced in the following paragraphs in the sequence in which they will be discussed within the major sections of Appendix B.

The treatment component of wastewater management can be accomplished in a variety of ways, depending on the treatment technology considered. Advanced biological and physical-chemical treatment processes utilize treatment plants as vehicles for wastewater renovation. Renovating via land treatment technology is also a viable alternative, offering many advantages. Industrial treatment systems consider wastewater flows

from the industrial community of the C-SELM area. Particular attention is paid to the two major industries, namely steel and petroleum. Wastewater generated by these industries will have to undergo pretreatment prior to discharging the effluent into the regional treatment plants or land treatment conveyance system.

One of the by-products of wastewater treatment is the solids-bearing residual known as sludge. The sludge management component consists of collection and ultimate disposal or utilization of stabilized residuals. Sludge is rich in nutritional ingredients and can be benefically utilized for agricultural purposes.

The wastewater flows generated within the C-SELM areas must be delivered to the advanced treatment facilities formerly mentioned. This is accomplished through two components of the management system, namely, the collection and the conveyance systems. The two components of water transportation take many forms, which are described in detail later in Appendix B, Section IV-D and IV-E.

The stormwater management component of wastewater has a particular significance in this study. Rain arrives at a given area in large quantities and at a variety of intensities and duration. The resulting stormwater runoff, although modified in intensity, cannot be treated at the range of normally encountered intensities for reasons of economies in the design of treatment facilities. It therefore must be held and released for treatment at rates that are commensurate with the capabilities of the treatment facilities. This holding, or storing, of stormwater is the basic principle of the stormwater management system.

Construction of stormwater storage and conveyance systems produces large quantities of rock and residual soils. The management of these two by-products is discussed in the Appendix B, Section F.

Renovated water is another product of the wastewater treatment system. It is used as a source of recreational flows in the streams and rivers of the region. It also is used to augment the flows of the navigable arteries which serve the commercial segments of the C-SELM area. Its NDCP quality is such that it can also serve as a source of potable supply. The reuse of renovated water is therefore a desirable system component of regional wastewater management.

The synergism component explores the mutually beneficial opportunities presenting themselves as a result of the establishment of regional wastewater management. Those opportunities include such possibilities as better land-use planning, promotion of recreational open space, agricultural stability and the use of land treatment storage lagoons as cooling ponds for power plants or pumped storage sites.

The non-structural management component discusses a variety of considerations such as control of soil erosion, water conservation, control of pleasure and commercial watercraft wastes, control of water pollution from solid waste landfills, phosphate detergent bans, and septic system characteristics.

These management component systems are discussed in detail in Appendix B, Sections II, IV, and VI.

I. INTRODUCTION

C. STUDY AREA DESCRIPTION

PHYSICAL FEATURES

Area

The study area encompasses 2,600 square miles in all or part of seven contiguous counties of the states of Illinois and Indiana. Counties included are Lake, Cook, DuPage, and Will in Illinois and Lake, Porter and LaPorte in Indiana. Figure B-I-C-1 shows the study area and its geographic boundaries. Study area boundaries primarily reflect principal drainage basin delineations. The only exception is the exclusion of a small upland drainage area at the headwaters of the DesPlaines and Chicago river systems in Wisconsin. This area is purposely omitted in order to limit the regional study to two states. The large areas in proximity to the C-SELM area can also be significant in a regional management study and are therefore shown in Figure B-I-C-1.

Topography

The study area is entirely within the central lowland physiographic province. A further subdivision places the study area almost entirely within the Great Lakes section of this province. Only a small area in the southwestern corner lies outside of the Great Lakes section. The Great Lakes section is further subdivided into two subsections, the Chicago Lake Plain subsection, and the Wheaton Morainal County subsection. The Chicago Lake Plain subsection extends from Winnetka on the north to the Indiana border on the south. It is bounded on the west by La Grange, Illinois and slopes gently to Lake Michigan on the east. This subsection is a low flat area interrupted by only a few low ridges. The Wheaton Morainal County subsection forms a concentric band to the west of the Chicago Lake Plain subsection. Topography associated with this subsection is identified by broad parallel ridges with numerous captive depressional areas which might contain lakes or swamps. A small part of the Kankakee Plain subsection of the Till Plain section can be found in a small portion of the study area in western Will County, Illinois.
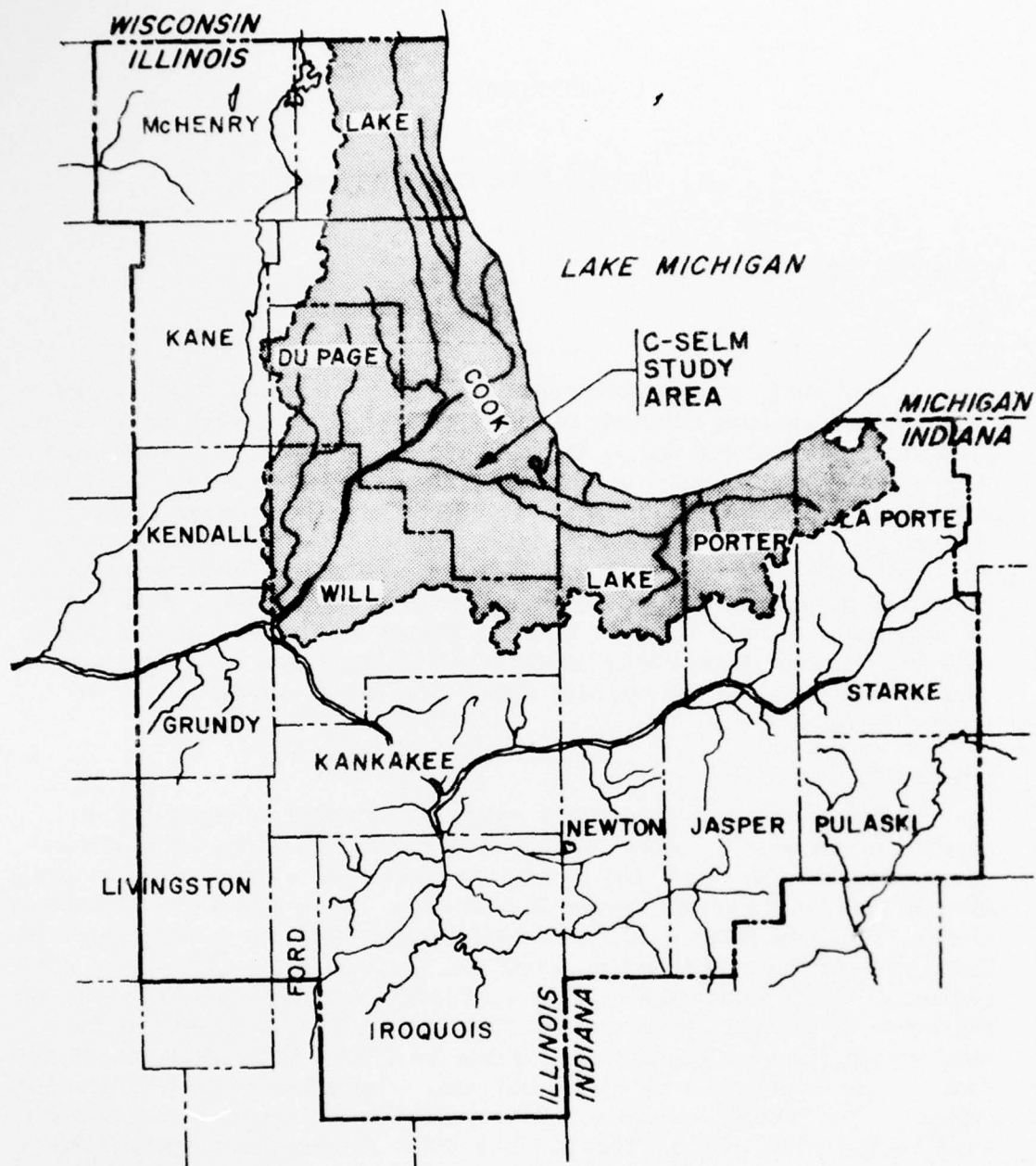
Figure B-I-C-I

Surficial features of the study area are a direct result of past glacial activity. Three glaciers, each advancing and retreating a number of times, once covered the area. Glacial drift from this action was deposited, and varies in depth from 0 to 400 feet. Bedrock outcroppings can be seen in portions of Cook and Will Counties, Illinois. Controlling geologic features of the area were also created by glacial action.

Each advance of a glacier rearranged the previous deposition and each retreat deposited another layer of material. The material which remained after the final glacial movement is of an extremely heterogeneous nature, and is known as glacial drift or till. Glacial end moraines formed ridges which parallel Lake Michigan. Glacial outwash deposits can be found between these moraines.

Topographic relief within the study area is low, with elevations varying from around 600 to 800 feet above sea level.

Drainage

Two main drainage patterns exist within the study area. In Illinois, most flows drain through the Illinois River system, while in Indiana most flows drain to Lake Michigan. The generally flat topography of the study area controls drainage patterns, which are usually not well defined. Historically, some 660 square miles within Illinois along Lake Michigan north of Chicago and around and south of the Lake Calumet region drained to Lake Michigan. Flow from this area has been reversed and diverted to the Illinois River system by the construction of man-made channels such as the Sanitary and Ship Canal and the Calumet-Sag Channel.

The major river systems in the study area are the Chicago, Des Plaines, DuPage, and Little Calumet Rivers. The Chicago, Des Plaines and Du Page Rivers flow generally to the south and west away from Lake Michigan. The Little Calumet River west of Hart Ditch generally flows to the west and subsequently out the Illinois River. The divide on the Little Calumet River near Hart Ditch will move with increases and decreases in flow volume. Therefore, at times, flow from the Hart Ditch may flow to the Illinois system. The remainder of the Little Calumet River system drains to Lake Michigan through another man-made channel, Burns Waterway.

Climate

The C-SELM area climate is predominately of a humid, continental nature. It ranges from warm in the summer to cold in the winter. The climate is definitely modified by its proximity to Lake Michigan. The average annual temperature is approximately 50°F; the average annual rainfall is 33.18 inches as recorded at Midway Airport, Chicago, based on a standard 30-year period of record. Snowfall produces about one-half of the precipitation during the winter months and one-tenth of the total annual precipitation (in inches of rainfall).

POPULATION

Total 1970 population for the C-SELM area was 7.2 million people. Population projections for the years 1990 and 2020 are 9.0 and 11.0 million, respectively. Detailed population information is referenced when it is used within the various sections of the Appendix, and will not be outlined at this time.

I.   INTRODUCTION

D.   STRUCTURE OF APPENDIX B

CONTENTS:

APPENDIX ORGANIZATION

The Appendix is divided into seven, roman-numeraled sections which outline the basis of design and cost of the regional wastewater managment system components presented in Appendix B, Section I-B, above.

Section I, Introduction, orients the reader to the concepts and goals of regional wastewater management and introduces him briefly to the Chicago-South End of Lake Michigan study area.

Section II, Summary of Design, presents a concise summary of design for each of the major components which make up the regional wastewater management systems. The section provides a general overview of system processes without detailing the component design parameters.

Section III, Flow Basis of Design; presents the detailed flow projections for the study area. Flows projected in the section include municipal (domestic, commercial) and industrial as well as stormwater flows.

Section IV, Component Basis of Design; describes in detail the basis of design for all component portions of the regional wastewater management systems.

Section V, Cost Estimation Methodology; outlines the methodology used to determine the cost associated with the individual components discussed in Section IV. In addition it presents a brief discussion of the economic analysis procedure used to determine individual alternative system costs which are presented in Appendix D.

Section VI, Component Basis of Cost; details the basis for cost of each of the wastewater management system components. Unit costs for system components are established for capital, operation and maintenance, and replacement items.

Section VII, Impacts of Management Systems, presents the impact of the management systems on selected resources such as, chemicals, energy, land, labor, etc. In addition, the section discusses the reliability and adaptability of system components.

B-I-D-1

DATA ANNEX ORGANIZATION

The data annex to this appendix is organized in a parallel structure to the formal appendix. The data annex contains more detailed supporting information.

APPENDIX LABELING

Page numbering and Figure and Table identification are referenced by a four place designation. An example of each is presented below:

Table or Figure Labeling and Referencing

Table or Figure   B – IV – A – 3

Identifies
reference as a   └Identifies table or figure number,
table or figure ┘   numbered consecutively from
                                           beginning of subsection.

                        Identifies subsection of section

                 Identifies section of appendix

            Identifies Appendix
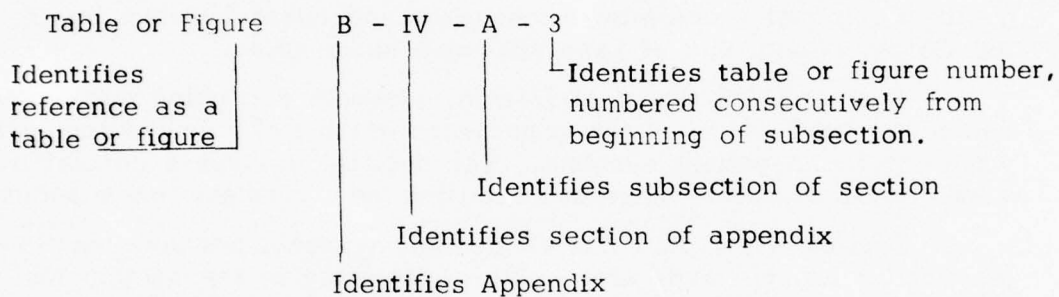
Page Numbering and Referencing

                    B – VII – D – 14

Identifies Appendix ┘   └Identifies page number, numbered
                                 consecutively from beginning of
   Identifies section of Appendix │   subsection

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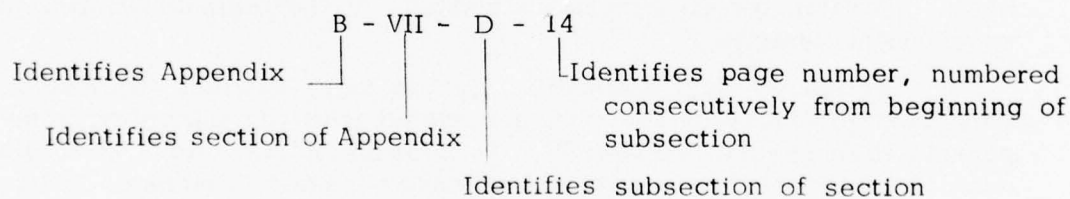
DATA ANNEX LABELING

Page numbering and Figure and Table identification are the same as the appendix, except the identification of the Data Annex begins with the letters "BA".

REFERENCES

Reference numbers for bibliographic references are listed chronologically at the end of appendix and appendix data annex subsections.

TECHNICAL APPENDIX B

SUMMARY OF DESIGN

II. SUMMARY OF DESIGN

A. REGIONAL TREATMENT SYSTEMS

INTRODUCTION

The aforementioned water quality goals are accomplished through the use of municipal wastewater treatment systems. As is currently practiced, these systems will treat wastes from domestic, commercial and industrial sources together with stormwaters which infiltrate into the sewer system. Due to the elaborate and extensive treatment processes necessary to accomplish the NDCP goal, economic considerations necessitate a treatment plant approach that is more regional than is presently practiced in certain C-SELM areas.

As presented in Appendix B, Section III-A, there presently exist some 132 municipal treatment facilities within the study area. The regional wastewater management alternatives presented in Appendix B utilize a base regional treatment system of 64 plants. This base system reflects present wastewater management plans of the various regional planning agencies operating in the C-SELM area. Of the 64 base plants, ten are proposed plants which are not presently in construction or operation. Thus the base impact of studying regional treatment systems is an abandonment of some 78 small existing treatment facilities.

As presented later in this section, it is anticipated that industries which currently provide on-site treatment prior to discharge to receiving waterways will find it economically advantageous to recycle the wastewater flows and send the blowdown flow to municipal regional facilities in order to meet the NDCP goals.

Provisions are also made in the regional treatment approach to treat urban-suburban stormwater runoff which presently flows directly into streams or bypasses treatment works.

In summary, the achievement of the water quality goals, as delineated by the NDCP goal, is accomplished in this study through municipal regional treatment works providing adequate capacity for handling, not only typical municipal flows, but also industrial and stormwater flows as well.

Treatment Technologies for Existing Standards

Wastewater treatment systems have been developed to meet existing effluent quality standards and, thus, serve as a reference base for the advanced wastewater treatment (AWT) systems which are designed to meet the NDCP goals. The treatment technologies associated with the present effluent standards are classified into five major types:

Type	Performance	Criteria
Type A	BOD - 20 mg/l, SS-25 mg/l	Illinois Dilution Waters 5:1
Type B	BOD - 10 mg/l, SS-12 mg/l	Illinois Dilution Waters 1:1
Type C	BOD - 4 mg/l, SS- 5 mg/l	Illinois Dilution Waters 1:1
Type D	BOD - 4 mg/l, SS- 5 mg/l, NH_3-N - 2.5 mg/l	Illinois portion of Chicago & Calumet River Systems
Type E	BOD - 20 mg/l, SS-25 mg/l, P - 80% Removal	Indiana flows to Lake Michigan

These technologies include conventional secondary treatment of wastewater and add-on advanced treatment components such as: mixed-media filtration for further biochemical oxygen demand (BOD) reduction and suspended solids (SS) removals; biological nitrification units for ammonia removal; and chemical precipitation for phosphorus removal. The flow diagrams for these treatment technologies are presented in Figure B-II-A-1.

Treatment Technologies for NDCP Goals

The wastewater management systems studied for the C-SELM area, incorporate three AWT technologies for achieving the NDCP goal. Two of the technologies, termed advanced biological and physical-chemical, require treatment plants and are referred to as treatment plant systems. The third technology, land treatment, utilizes significant amounts of land for the advanced treatment of wastewater and is referred to as the land treatment system. Presented in Figure B-II-A-2 are schematic flow diagrams of these three AWT technologies together with conventional secondary (activated sludge) wastewater treatment. The components of the three AWT systems presented in Figure B-II-A-2 are selected on the basis of such considerations as costs, environmental impacts and treatment performance capabilities necessary for the attainment of the NDCP goals.

BOD$_5$-20 mg/l
SS-25 mg/l

Type A

SEC

RECEIVING
STREAM

BOD$_5$-10 mg/l
SS-12 mg/l

Type B

SEC

FILT

RECEIVING
STREAM

BOD$_5$-4 mg/l
SS-5 mg/l

Type C

SEC

FILT

RECEIVING
STREAM

BOD$_5$-4
SS-5
NH$_3$-N - 2.5

Type D

SEC

NIT

FILT

RECEIVING
STREAM

BOD$_5$-20
SS-25
P-1

Type E

SEC

80%
P

RECEIVING
STREAM
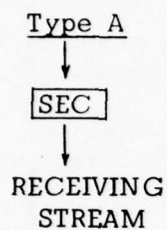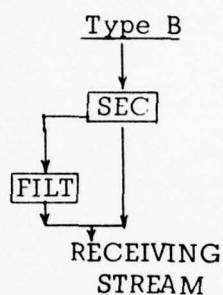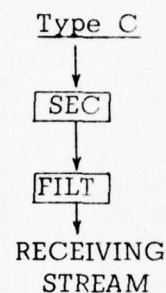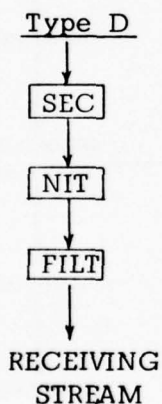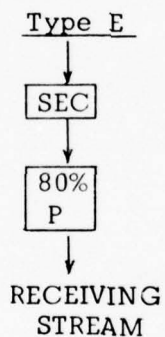
LEGEND

BOD$_5$ = Biochemical Oxygen
            Demand (5 Day)
SS = Suspended Solids
NH$_3$-N = Ammonia Nitrogen
80% P = 80% Phosphorus Removal
SEC = Biological Secondary System
NIT = Biological Nitrification
         System
FILT = Mixed Media Filtration

Figure B-II-A-1

TREATMENT TECHNOLOGY FLOW DIAGRAMS
TO MEET EXISTING EFFLUENT STANDARDS

LAND TREATMENT

WATER RESOURCE

→ DOMESTIC AND INDUSTRIAL USE

SLUDGE TREATMENT AND DISPOSAL

↓ COLLECTION AND TRANSPORT

↓ DEGRITTING

↓ BIOLOGICAL TREATMENT

↓ STORAGE AND DISINFECTION

↓ SPRAY IRRIGATION

↓ LIVING FILTER

↓ DRAINAGE COLLECTION AND MONITORING

↓ AERATION

↓ DISCHARGE

CONVENTIONAL TREATMENT

WATER RESOURCE

→ DOMESTIC AND INDUSTRIAL USE

SLUDGE TREATMENT AND DISPOSAL

↓ COLLECTION AND TRANSPORT

↓ DEGRITTING

↓ PRIMARY SETTLING

↓ AERATION

↓ SECONDARY SETTLING

↓ CHLORINATION

↓ MONITORING

↓ DISCHARGE

RECEIVING WATER

Figure B-II-A-2 (Continued)

FLOW DIAGRAMS FOR TREATMENT TECHNOLOGIES

B-II-A-5

LAND TREATMENT

WATER RESOURCE

DOMESTIC AND
INDUSTRIAL USE

SLUDGE TREATMENT
AND DISPOSAL

COLLECTION AND
TRANSPORT

DEGRITTING

BIOLOGICAL
TREATMENT

STORAGE AND
DISINFECTION

SPRAY
IRRIGATION

LIVING
FILTER

DRAINAGE COLLECTION
AND MONITORING

AERATION

DISCHARGE

CONVENTIONAL TREATMENT

WATER RESOURCE

DOMESTIC AND
INDUSTRIAL USE

SLUDGE TREATMENT
AND DISPOSAL

COLLECTION AND
TRANSPORT

DEGRITTING

PRIMARY
SETTLING

AERATION

RECEIVING WATER

SECONDARY
SETTLING

CHLORINATION

MONITORING

DISCHARGE

Figure B-II-A-2 (Continued)

FLOW DIAGRAMS FOR TREATMENT TECHNOLOGIES

B-II-A-5

II. SUMMARY OF DESIGN

B. INDUSTRIAL TREATMENT SYSTEMS

INTRODUCTION

C-SELM Industry

C-SELM industries can be divided into two groups, those which discharge their wastewater into surface waters and those which are tributary to regional, municipal plants.

The surface dischargers can be further divided into two groups, the critical industries and the non-critical industries. The critical industries in the C-SELM area are the steel industry and the petroleum industry. At the present time these two industries discharge 90% of the industrial wastewater to C-SELM.

Purpose of This Section

The purpose of this section is to examine closely the detailed treatment systems associated with these above-mentioned industries for various levels of wastewater treatment including present practice, present standards and NDCP Standards. From this detailed analysis will come a cost estimate for each of the various levels of treatment for each of the two industries. Finally, the cost increment to move from current standards to NDCP standards will be developed for each of the two industries and will be used to model and project the similar incremental cost for the non-critical industry agglomerate. The sum of these incremental costs integrated across that portion of C-SELM industry not tributary to, and therefore not financially supporting, regional treatment systems can then be combined with the costs of the regional treatment systems to develop the total cost impact of NDCP effluent quality standards.

General Descriptions

Recycle of reclaimed wastewater, the recovery of solids and the reuse of partially reclaimed wastewater, particularly as a source of industrial process water, are concepts that are central to a resourceful management of industrial wastewater.

Cooling flows are recycled in order to minimize problems with thermal pollution impacts on natural watercourses and to maximize benefits from the available water resource.

Critical Industry Analysis

The resulting detailed cost increment, projected for the steel and petroleum industries as a result of adopting NDCP effluent quality standards, was demonstrated to be actually less than zero signifying that, effectively, there were major cost savings that counter-balanced the increased cost of treatment. These savings result from opportunities for increased recycling of process water thus diminishing the blowdown flow that ultimately must be treated, and to the extent that economy-of-scale effects are achieved by utilizing large AWT regional facilities for ultimate treatment rather than on-site, small-scale industrial treatment facilities.

II. SUMMARY OF DESIGN

C. SLUDGE MANAGEMENT SYSTEMS

INTRODUCTION

The treatment of C-SELM wastewater results in two end-products: the treated effluent and the solids, or sludge, removed during treatment. The treatment and disposal of sludge is a major design consideration of this wastewater management study.

The problem of dealing with sludge is complicated. The solids content of sludge represents only a small percentage of its total weight, with the rest being water, both cell tissue water and supernatant water.

The sludge management system involves the collection and treatment of raw sludge followed by its ultimate disposal or, preferably, its beneficial utilization. The final disposal of some or all parts of the sludge must ultimately be on the land, because the inland location of the C-SELM study area makes the use of other disposal methods impractical.

The generalized sludge management system discussed in this brief introduction is common to all of C-SELM wastewater management alternatives. The types of sludge generated, the types of sludge utilization, and the sludge management techniques employed for this study are introduced in the following paragraphs.

SLUDGE TYPES

There are two groups of sludges which can result from wastewater treatment processes; these are biological or organic sludges and physical-chemical sludges. Sludges produced from the biological treatment of wastewater contain primarily organic solids. Sludges and ash resulting from the physical-chemical treatment of wastewater contain largely inorganic solids.

Biological Sludges

There are three types of biological sludges which result from C-SELM treatment alternatives. They are the conventional, the advanced biological and the land treatment sludges. Conventional and

land treatment sludges have a high concentration of decomposable organic matter. Advanced biological sludge contains a high concentration of organic matter along with the ash resulting from the lime recalcination process associated with the advanced biological treatment system. The organic matter in these raw sludges, normally described in terms of volatile solids, can produce offensive odors if allowed to decompose in an unregulated manner. To prevent this, anaerobic digestion is used to stabilize the organic matter. The C-SELM biological sludges are expected to be 6% total solids by weight with the balance being water.

The land treatment and conventional biological treatment processes can be expected to produce 0.77 dry tons of anaerobically digested sludge per million gallons of sewage. Treatment plants utilizing the advanced biological treatment process can be expected to produce 1.64 dry tons of anaerobically digested sludge and ash per million gallons of sewage inflow. These yield figures include grit.

Physical-Chemical Sludges

Raw physical-chemical sludges contain inorganic precipitates formed during the treatment process along with the organic solids contained in raw sewage. This sludge is incinerated during the lime recalcination process, an integral part of the physical-chemical treatment plant technology. The resulting inorganic ash from this process is in the final form of physical-chemical sludge. For pipeline transmission this ash is mixed with water so that the resulting mixture contains no more than 10% total solids by weight.

The yield of physical-chemical ash is expected to be 1.13 dry tons per million gallons of raw sewage treated.

SLUDGE UTILIZATION OBJECTIVES

Ultimate disposal of sludge generated as a by-product of sewage treatment is accomplished by application to the land. Two types of land application of sludge are considered, agricultural utilization and land reclamation.

Agricultural Utilization

The agricultural utilization of sludge employs sludge as a fertilizer or soil conditioner providing important nutrients for plant growth. Sludge is applied to the agricultural land at a controlled rate on a yearly basis. The sludge application system is a permanent installation.

Both biological sludges and physical-chemical sludges can be applied to the land for agricultural utilization. Biological sludges are used for their acidity control and soil conditioning values.

As presented in detail in Data Annex B, Section C, the optimum sludge application rate for the agricultural utilization of conventional and land treatment sludges over a 50-year period is 13.5 dry tons/acre/year. For advanced biological sludges, the corresponding rate is 28.8 dry tons/acre/year. The sludge application rate is adjusted so that the total nitrogen applied to the land is equal to the nitrogen uptake of crops plus the nitrogen lost through volatilization and soil denitrification. Increasing the organic nitrogen content of the topsoil is ignored as a means of consuming additional nitrogen, as a limit could be reached in this process before the end of the design 50-year period. Thus a maximum crop yield can be expected without a simultaneous groundwater pollution problem. The 50-year accumulations of these sludges is not expected to produce excessive accumulations of the associated heavy metals.

The optimum sludge application rate for the agricultural utilization of physical-chemical sludge is 1.73 dry tons/acre/year. This application rate is determined by the alkalinity of the sludge.

Land Reclamation

The land reclamation approach assumes the application of biological sludges to strip-mined areas in Indiana and Illinois at a controlled rate during a short period of time. The alternative of using these sludges to restore the original organic nitrogen content of heavily cropped soils is not considered in this study, but is a possibility if a future market for this use becomes much larger than at present. As the soil in the strip-mined areas contains only limited amounts of organic matter or humus, the application of sludge serves to increase the humus content and the fertility of the soil, stimulating the growth of grass or trees for recreational uses. The physical-chemical sludge

does not contain nitrogen or humus value, and therefore is not used for land reclamation applications. The sludge application system installed in a land reclamation area will be temporary because of the short duration of the operation.

The optimum sludge application rate for the land reclamation utilization of advanced biological sludge is 213 dry tons/acre. The optimum sludge application rate for the land reclamation utilization of conventional and land treatment sludges is 100 dry tons/acre. The nitrogen balance in these systems is also assumed to control their application rates.

In the land reclamation utilization of sludge, the sludge application rate is based on the amount of nitrogen remaining in the sludge after air stripping has removed most of the ammonia nitrogen. The sludge is made alkaline for the air stripping of ammonia, and the high alkalinity is useful in counteracting the acidity ordinarily found in strip-mine areas. As most of the remaining nitrogen in the sludge is in the organic form and the rate of mineralization of organic nitrogen is low, a high-rate, short-duration application of sludge will not produce any groundwater pollution problems. On the contrary, the grass cover made possible by the sludge application greatly improves runoff water quality.

ALTERNATIVE NITROGEN CONTROL TECHNIQUES

In each of the foregoing alternatives, the amount of nitrogen in the sludge was assumed to control the amount of sludge which could be applied at any one time, the nitrogen content of each type of sludge was assumed to have a fixed value, and no accumulation of nitrogen in the soil-building processes was assumed to occur. Such an approach results in equal treatment for all alternatives.

However, the actual nitrogen content and the amounts of each of the different kinds of nitrogen present in a given sludge will be different from the corresponding amounts in some other sludge. In any actual operation, such differences would be taken into account.

Furthermore, recent research in using large amounts of an old sludge, without ammonia stripping (up to 182 dry tons per acre) on farms in central Illinois have demonstrated the absence of any measurable nitrate or ammonia pollution of ground or surface waters, even though calculations of the type used in this study would indicate that such problems should have resulted. Evidently, the mathematical model used herein is conservative. It is likely that the continuing research

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into the amount of sludge which can be successfully applied without ammonia stripping or some other form of nitrogen control will permit much larger short-duration applications.

When viewed from the point of view of benefits to be achieved in the plans considered in this study, however, there does not appear to be any incentive to use higher application rates than those proposed here. For example, over a period of 50 years it is planned to apply 1,400 dry tons per acre of the advanced biological sludge. Instead of applying 28.8 tons each year for 50 years, 288 dry tons could be applied per acre each year for 5 years to 10% of the total land area involved; application would then proceed to the next 10% for the following 5 years, and so forth. This technique would probably not result in the maximum possible utilization of the nitrogen available for crops, even though natural denitrification and soil-building processes might eliminate problems of nitrate or ammonia pollution.

With regard to the land reclamation approach, the maximum benefit is achieved with the reclamation of the maximum possible amount of strip-mined land. The quantity of such land available for reclamation is likely to be greater than the amount of sludge which would be available to reclaim it. Therefore, if 100 dry tons per acre can reclaim such land, a larger amount should not be used. The incentive here is to reclaim the maximum number of acres.

SLUDGE MANAGEMENT TECHNIQUES

The land application of C-SELM sludge will be carried out using the following management techniques for each treatment system alternative.

In the conventional biological treatment system, the sludge is collected and stabilized using high-rate anaerobic digestion. Following digestion, the sludge is lagooned for dewatering at the treatment plant site. After dewatering to the 6% total solids level, the sludge is transported to a land application site where it can be used for agricultural purposes.

In the advanced biological treatment system, the biological sludge is collected and stabilized using high-rate anaerobic digestion while the physical-chemical sludge is collected and incinerated to

ash in the lime recalcination process. The digested sludge and ash are combined and the resulting sludge at 6% total solids is transported to a land application site where it can be used for agricultural or land reclamation purposes.

In the physical-chemical treatment system, the sludge is incinerated to ash in the lime recalcination process. The ash is then mixed with water to a concentration of 10% total solids and is transported by pipeline to a land application site for use as a soil conditioner or for pH control. The alternative of landfilling the dry ash was not considered to be a viable alternative because of potential leaching problems and because there would be no beneficial use of the alkalinity of this sludge.

In the land treatment system, the solid wastes are conveyed with the wastewater to the land treatment sites where, after biological treatment, they are stabilized by anaerobic digestion on the bottom of the land treatment storage lagoons. After a period of years, the digested sludge is dredged from the bottom of the lagoon and applied to adjoining agricultural utilization areas or transported to land reclamation areas.

II.   SUMMARY OF DESIGN

D.   STORMWATER MANAGEMENT SYSTEMS

STORMWATER MANAGEMENT

Definition:

Water arrives at the earth's surface as rainfall or snowfall. Only a portion of this rainfall or snowfall finds its way to sewers or streams, while other portions return to the atmosphere through evaporation and transpiration, and another portion infiltrates the ground and enters deep aquifers. The portion which finds its way into stormwater systems, combined flow systems, and, via overland flow to channels, streams or rivulets is called runoff. Runoff may or may not be stored in natural lakes or swamps. In an urbanizing area, such natural storage is gradually eliminated. Stormwater management involves the interception of runoff, storage of it to regulate instantaneous flow rates, and its appropriate treatment.

The stormwater runoff, may be contaminated by contact with the surface of the earth or by passage through a polluted atmosphere. Contamination takes many forms associated with the great variety of surfaces that cover the face of our planet, and with the variety of atmospheric pollutants common to urban areas. Farmland, crop land, pastures, forests, rural dwellings, sprawling suburbs and large areas of pavements and roofs of our cities add many pollutants to the precipitating water which, in its passage through the atmosphere, may have picked up other pollutants. In order to prevent these contaminants from entering our streams and rivers, we must manage the stormwater by capturing and treating it.

Storage Needs

From the records of past years, an average of 33 inches of rainfall per year can be expected in the C-SELM area. Detailed studies employing 21 years of rainfall records and measurements of associated runoff indicate an average runoff of 19 inches from urban areas. Suburban and rural areas are estimated to contribute 12 inches and 10 inches, respectively.

Historical records of 21 years of precipitation indicate that it would be uneconomical to provide storage for <u>all</u> the runoff. On the other hand, sufficient storage can be provided at reasonable cost to contain about 98% of runoff. This storage is also adequate to prevent spillage to streams of pollutant loads which would otherwise prevent the achievement of proposed quality goals. Once the storm-water is in storage, it is possible to pump it out for treatment at a desired pump-out rate. By varying the assumed pump-out rate, a cost curve of associated costs of storage and treatment can be calculated. This procedure allows selection of the most economical storage volume for different land use areas and permits an optimization of management costs. Results of this optimization, which is described in Appendix B, Section IV-D, are presented below:

     Urban Areas: 2.5" storage with .004 cfs/acre pump-out rate

     Suburban Areas: 2.85" storage with .002 cfs/acre pump-out rate

     Rural Areas: 2.5" storage with .002 cfs/acre pump-out rate

Storage Systems

The urban area, (consisting mainly of the City of Chicago and several adjoining suburbs) is managed by the comprehensive storage systems contemplated in the Chicago Underflow Plan using a single, large quarried site in the McCook-Summit area with 57,000 ac-ft capacity, and two smaller sites with additional capacity of 5,800 ac-ft.

Existing suburban areas are served by a large number of local storage sites of two categories. One of the types contemplated is shallow-pit storage, with from 20 to 1000 acres in surface area, and with depths ranging from 15 to 20 feet. These sites are located on presently available, low open land. The other type of storage (in areas of dense population distribution) is mined in the underlying rock formations. This is a more expensive type of storage and is used only when absolutely necessary.

Rural areas are served by retention basins placed throughout the rural region; each sub-watershed served ranges in size from roughly 1,500 acres to 6,400 acres. Further discussion of storage for rural areas is presented in the Rural Stormwater section in this Summary of Design.

Collection Systems

Stormwater is intercepted and directed to these storage sites through urban, suburban, and rural collection systems which may collect either combined flows in the urban areas, or separate, municipal and industrial, or stormwater flows in suburban areas, as well as overland runoff in the rural areas. All collection systems terminate at storage sites, treatment plants, or access points. For the purposes of this study, the urban collection system is assumed to be in existence and its modifications and interconnections with storage are those contemplated in the Chicago Underflow Plan. The suburban collection systems are assumed to be partly in existence and partly to be developed. Discussion of rural collection systems is presented in the Rural Stormwater section in this Summary of Design.

RURAL STORMWATER

Philosophy of Design

Stormwater runoff takes two forms: (1) overland flow, and (2) infiltrated or groundwater flow. In rural areas, the larger portion of total rainfall and snowfall is infiltrated. Such water arrives at the stream in a clean condition. Water which runs overland to streams may be contaminated by the materials it picks up enroute. Stormwater flows in the rural area are managed in two ways. First, a concentrated effort is made to increase infiltration. Secondly, improved natural drainage channels are used to convey flows to manmade storage or retention basins located within the watersheds. The captured flows are temporarily stored, and later infiltrated into the soil, imitating the natural infiltration and storage. These two mechanisms are discussed below.

Rural Land Management

The proposed rural stormwater management starts with the implementation of a full range of standard, United States Soil Conservation Service (SCS) practices designed to reduce the rate of surface runoff and the rate of soil erosion. Two practices are proposed, one dealing with tillage operations and a second with crop residues.

Tillage operations are kept at a minimum, with the ground surface left as rough as possible to provide increased absorption of rainfall and reduced erosion. Crop residues, after harvest, are left on the surface to protect the soil from erosion during the nongrowing season.

The combination of minimum tillage and crop residue buildup not only reduces erosion, but it greatly increases the available surface storage. Runoff from these properly managed areas flows naturally overland with a reduced sediment load to established drainage channels.

Rural Stormwater Surface Runoff Collection

An improved natural drainage system is a very important link in the proposed management of rural watersheds. These natural drainage ways are shaped and a grass cover established to provide a relatively erosion-free movement of runoff. In addition, plastic tile drainage systems are installed parallel to the centerlines of these grassed waterways to provide adequate subdrainage and to maintain the necessary aerobic conditions for a healthy growth of grass.

The flow conveyed by such an improved natural grassed waterway is assumed to have a high nutrient content. It is, therefore, intercepted by another grassed waterway channel placed parallel to the existing main stream to prevent the polluted flows from reaching that stream. This intercepting grassed channel leads to a temporary holding basin which attenuates variations in flow rates.

Rural Runoff Storage

Storage reservoirs are provided in natural waterways which have historically been associated with intermittent flows. Storage is not provided on perrenial streams. Collected flows are routed to these reservoirs, from which the water is withdrawn for treatment.

Storage volume requirements for the rural management areas are based upon optimization studies which are described in Appendix B, Section IV-D.

Rural Runoff Treatment

The water which is temporarily stored in reservoirs is infiltrated prior to release to adjacent streams. Infiltration of these stored flows is accomplished by the use of center-pivot spray irrigation machines located on existing agricultural land near the reservoir. This system accomplishes treatment according to the "living filter" concept.

Rural Runoff Reclamation

The collected surface runoff, which is infiltrated through spray irrigation, is moved to the closest natural water course through plastic

drain tiles which also maintain an adequate aerobic zone for crop growth. A portion of runoff thus recovered is diverted to a nearby well for injection into the local aquifer. The aquifer serves as a long-term storage of renovated water, which can be pumped-out for potable water needs during periods of drought.

Reservoir Site Selection

Site selection was investigated by studying three specific watersheds divided into 22 subwatersheds. The 22 subwatersheds cover over 66,000 acres, or approximately 8% of the total C-SELM area. These watersheds were selected to serve as models, representative of the entire C-SELM area. A detailed discussion of the characteristics of these subwatersheds and of the proposed management system is presented in Appendix B, Section IV-D.

Rural Management System Flexibility

The proposed rural runoff management system will enhance not only the existing rural land but also a future suburban development which could occur. The grassed waterway system paralleling the existing perennial streams provides recreational greenbelt corridors for access to many streams. Storage reservoirs could provide permanent pools which could also be used for recreational purposes.

The current trend in the C-SELM area has been to require developers to provide stormwater runoff storage. Management systems as proposed here would provide the developer with attractive storage and conveyance facilities which he could integrate directly into his planned development.

II.   SUMMARY OF DESIGN

E.   CONVEYANCE SYSTEMS

Conveyance systems transport wastewater flows to regionalized treatment facilities from storage sites or access points. Access points are all former treatment sites which are eliminated during the process of regionalizing the treatment facilities. Conveyance systems terminate at either regionalized treatment plants or land treatment sites. Physically, conveyance systems consist of pumping stations and pressure lines, or gravity tunnels. Gravity tunnels are used when large quantities of water must be transported over great distances.

DESCRIPTION OF CONVEYANCE SYSTEMS

Proposed C-SELM conveyance systems transport wastewater from three sources: stormwater flows; municipal and industrial flows; and combined-sewer flows.

DESIGN BASIS OF CONVEYANCE SYSTEM

Each system is designed to achieve the required conveyance capacity at minimum total cost. Minimum velocities and slopes were used where these would govern the design.

PHYSICAL LOCATION OF CONVEYANCE SYSTEMS

Conveyance pressure line and tunnel systems are laid out along and within public rights-of-way to avoid the necessity of obtaining these rights from private parties. Tunnel alignments are checked with local geological configurations for best routings and depths available.

GEOLOGIC CONSIDERATIONS

A section on geologic characteristics of the C-SELM area is included in Appendix B, Section IV-E and discusses important features of rock formations encountered.

II.   SUMMARY OF DESIGN

F.   ROCK AND RESIDUAL SOIL MANAGEMENT SYSTEMS

DEFINITION

The rock and residual soil management systems comprise all of the means of moving the rock from tunnel and storage excavation and the residual soil from storage, force main and sewer excavation from the point of origin to the point of final use or disposal. The means include provisions for loading, transport, transfer, unloading, and final placement. The actual excavation is included in the construction of the conveyance and storage facilities.

The three types of material that are differentiated for the purpose of management are mined rock, moled rock and residual soil. Mined rock includes the dolomite bedrock that is removed by quarrying techniques in pit or room-and-pillar construction. Moled rock is the dolomite bedrock that is removed from tunnels that are constructed by the use of mole tunnelling machines. Residual soil is any material other than dolomite bedrock that is removed and not replaced during excavation of pit storage basins, and construction of sewers and force mains.

DESCRIPTION OF MATERIALS

The three materials differ in composition, quantity and location, each of which affects the opportunities and limitations of the management programs.

Mined Rock

Mined rock comprises the largest quantity of material requiring management. Excavation of the McCook-Summit storage basin, which is a component of the stormwater management system for all of the wastewater management system alternatives, requires the management of approximately 275,000,000 tons of dolomite rock. If this is accomplished within a ten-year period, the annual output from this location approximates the annual production [1]/of all of the rock quarries in the Chicago metropolitan area combined.

The other sources of mined rock are the suburban pit storage sites, which are deep enough so that it is assumed that one-half of their total volume is excavated from bedrock. The estimated quantity of rock from these sources is 31,000,000 tons, the source being distributed throughout the more densely populated suburban area, where deep pits are necessitated because of space limitations.

Mined rock is excavated by the drill-and-blast method that is used in quarrying operations. The resulting material ranges in size from pieces the size of an automobile to fines. Crushing is required before loading and transport. With sorting and grading, this material is the same as that used for aggregate and stone in any quarry, provided the rock quality from the particular location is satisfactory.

A density of 2.16 tons per cubic yard is used to determine the weight of rock excavated. The final density, after placement and compaction at the disposal site, is assumed to be 1.8 tons per cubic yard.

Moled Rock

All of the stormwater, conveyance and reuse tunnels are constructed using "mole" boring machines. The tunnel sizes for which mole machines have been used range from 6 to 36 feet in diameter. [2] The maximum size of the tunnels in this study is 42 feet in diameter, and it is anticipated that a mole machine capable of tunnelling such a diameter of hole would be available.

Conventional practice in excavating tunnels provides for constructing a shaft for access of men and machines and for removal of rock about every five miles, as well as one where two tunnels join or where a diameter change dictates access for a machine.

The market value of moled rock is much less than that of mined rock because of its limited commercial use. The size ranges from about 4-inch pieces to fines. The preponderance of fines makes large quantities of moled rock virtually without commercial value.

The same densities are assumed for the moled rock as for the mined rock: 2.16 tons per cubic yard before excavation and 1.8 tons per cubic yard after placement and compaction at the disposal site.

Residual Soil

Residual soil, as the term is used here, consists of the variety of materials that overlie the bedrock in the region. Also included in this classification is the liquid sludge that is contained in lagoons at the site of the McCook-Summit storage basin. It is assumed that the sludge will be spread on, or mixed with, the residual soil at a sludge utilization site to accomplish soil enrichment. An initial density for the wet sludge of 0.9 tons per cubic yard, a concentration of 22 percent solids by weight, and a dry volume of 12 percent of the wet volume are used to determine the approximate quantities of the sludge. The 12 percent figure assumes a blending of sludge with residual soils at the sludge application site.

The remainder of the residual soil is assumed to have a density of 1.62 tons per cubic yard both before excavation and after placement and compaction at the disposal site. There may exist a commercial market for the soil and/or sludge.

The sources of the residual soil are the McCook-Summit storage basin, the O'Hare storage basin, some shallow tunnels in the O'Hare area, rural and suburban storage basins, storage basins at rural treatment plants, shallow gravity sewers and force mains to link the rural and suburban treatment plants, and force mains for reuse flows.

MANAGEMENT OPPORTUNITIES

The mined rock and residual soil must be managed so as to minimize the cost of disposal, the disruptive effects of transport, and the adverse impacts in the disposal areas. Proper management is essential particularly because much of the materials originates in the most densely populated areas of the region.

The materials present a wide range of opportunities, as well, for beneficial use within the region. These opportunities may be broadly classified as recreational, reclamation, or commercial. Selection of the final management procedures will be made on the basis of cost, compatibility with regional goals, disruptive effects on the communities, environmental effects, and other factors. Some examples

of possible management options are included here to illustrate the potentials for constructive use of the materials. Many other possibilities undoubtedly exist, and it is not intended here to recommend any one option for implementation.

Recreational Opportunities

Mountain Landscape. The rock and soil from the urbanized area is sufficient to construct a mountain approximately 430 feet in height and 600 acres in area at its base. This mountain would be suitable for skiing, hiking, tobogganing, picnicing, nature study, wildlife management, and other recreational uses.

Recreational Islands in Lake Michigan. By utilizing the materials excavated in the urban area for fill, and with extreme care for environmental and ecological effects, it would be possible to construct islands along the Illinois and/or Indiana shoreline for recreational use. With proper design, the islands may have the added benefits of protecting the shore from erosion and for providing a protected area for recreational boating. Sufficient material will emanate from the urban areas to construct approximately 2,000 acres of islands, assuming an average water depth of 30 feet. Construction of polders rather than islands would enable substantially more usable area to be constructed.

Other Open Space Landscaping. The extensive nature of the tunnel systems allows opportunities for relatively modest quantities of materials to be utilized by many communities to construct recreational landscapes in diverse open space areas. This could take the form of modest walking or tobogganing hills in city parks, larger hills in forest preserves or suburban areas, or landscaping of abandoned quarries to make them suitable for recreational use. These options have the advantage of making use of the materials nearer their point of origin, thus decreasing cost and disruption due to transport.

Reclamation Opportunities

Breakwaters or Other Lake Michigan Shoreline Protection. Materials would be available to protect or stabilize extensive lengths of shoreline from erosion. No configurations or construction problems are analyzed here, although the costs and transportation methods may be similar to those for the recreational islands.

Harbor Construction and Other Filling. Materials from this system could be used for construction of Calumet Harbor and other public or private landfilling that is in the public interest.

Renovation of Strip Mined Land. Extensive areas of land which has been mined and left in an unusable state lie within Grundy, Will, LaSalle, Livingston and Kankakee Counties. The dolomite rock and overburden could be transported to these areas and used for fill, for drainage improvement and for pH control within the strip mined areas.

Commercial Opportunities

Sale as aggregate or stone in competition with existing producers. This option is considered very unrealistic for any significant portion of the material management. The quantities of rock to be managed, are larger than the entire market for stone in the metropolitan area. This option would destroy the stone and aggregate industry that exists today.

Hiring of aggregate industry to mine the rock and store or sell it as the market demands. The market for limestone rock from the three existing quarries in the McCook-Summit area is about 6,000,000 tons per year. Since it can be reasonably assumed that the major share of rock at the McCook-Summit site is of marketable quality, this same amount of rock could be placed into the market from the C-SELM operations. Negotiations with the operators of the quarries in the area would be necessary to obtain the best cost for the taxpayers while not jeopardizing the private enterprise within the aggregate industry.

The time period of construction is critical in the evaluation of the impact of marketing upon the cost of rock management. Stockpiling of mined rock may be possible in existing quarries, but would involve additional handling which would tend to offset some of the advantages of marketing the rock. It is probable that commercial use will be made of less than half the mined rock.

Interviews with knowledgeable people within the aggregate industry led to several conclusions about the McCook-Summit storage basin. Due to the size, shape and location of the basin, the techniques available today for mining and handling the rock would probably be inadequate to meet the kind of construction schedules being discussed. However, assuming construction schedules could be met, the impact of this construction upon the existing aggregate and stone industries would be enormous, necessitating careful planning to avoid adverse economic effects upon private industry. There is, however, a willingness for cooperation by the industry to engage in discussion and negotiation which would benefit both the public and the industry.

Trading an existing quarry in the McCook area for the storage reservoir site. It is possible that this option would result in a substantial savings in quarrying and disposal costs, but the feasibility can not be determined without an extensive analysis and negotiation.

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II. SUMMARY OF DESIGN

G. REUSE SYSTEMS

INTRODUCTION

The reuse of high quality reclaimed water is a logical exten-
sion to the overall regional wastewater management concept. Water
is a limited resource. This is particularly true in the C-SELM area,
even considering the proximity of Lake Michigan. Groundwater sup-
plies are not being replenished as rapidly as they are being drawn
down, and the difference between withdrawal and recharge is growing
each year. Probably the most important reuse consideration, however,
is the current United States Supreme Court limitation on withdrawals
from Lake Michigan by Illinois residents. The Supreme Court limita-
tion of 3200 cfs places a definite restriction on the quantities of
water available to the densely populated urban and suburban areas of
Northeastern Illinois. However, the present diversion limitation was
established on an open-ended basis. The State of Illinois can peti-
tion to have the diversion limit increased if it can show that all
available supplies are being prudently managed and that inadequacy
of the available supply is imminent.

An important constraint on this present study is the avoidance
of the return to Lake Michigan of water withdrawn from it by Illinois
municipalities and industries. Water withdrawn from the lake by
Indiana municipalities and industries is assumed to be returnable to
it after use and treatment. These assumptions merely reflect current
practice.

The importance of this constraint lies in the high cost of stor-
age of treated water for reuse if Lake Michigan cannot be used for
this purpose. The cycle of wet years and dry years occurs over many
years. If water resulting from C-SELM storm runoff is to be captured,
treated, and used in a beneficial manner, it must be stored until a
later time when there would be a demand for it. Such storage is
likely to require long-term storage -- perhaps years.

Furthermore, the Great Lakes, themselves, undergo cycles of
high and low stages, with Lakes Michigan and Huron fluctuating over
a range of six feet or more. During periods of high stages, as at

the present time (1973), damage from accelerated beach erosion oc-
curs along the shoreline. During years of low level, damages occur
from insufficient navigational depths and from lack of flow for hydro-
electric power generation. If these damages from both high and low
lake levels are to be avoided, flows into and from the lakes must
be managed. If this is to occur, present patterns of withdrawal and
return of Lake Michigan water by residents of Illinois and Indiana
must be altered.

Investigation of such possible desirable changes of present
practices is beyond the scope of this study. If such changes were
to occur, significant cost reductions could be achieved for the reuse
operations envisaged in this study.

Reuse of water can serve many needs, but in this study two
uses have been selected. First, potable water is made available to
critical need centers to meet forcasted deficiencies. Potable water
deficiencies resulting from extreme lowering of the groundwater through
excessive pumping has removed the base flow from many streams, re-
ducing their recreational and aesthetic value. The second reuse pro-
vision is for the maintenance of adequate recreational streams flows.
This provision also includes a reuse allowance for navigational needs.
In order to properly evaluate these two reuse provisions, and to grasp
fully their impact on the existing water resource, two options for re-
use implementation have been established. The first option assumes
continuance of the current 3200 cfs limitation on withdrawals from
Lake Michigan by Illinois, and unlimited withdrawal for Indiana. The
second option allows unlimited withdrawals from Lake Michigan by
both Illinois and Indiana.

The two reuse provisions are discussed below in general
terms.

POTABLE REUSE

Potable water deficiencies are forecast mainly in the suburban
areas which have historically been the largest users of groundwater
supplies. For this reason, two potable need areas have been identi-
fied: (1) present groundwater service areas, and (2) present Lake
Michigan service areas. Forecasted potable deficiencies for these
areas are presented in Appendix B, Section IV-G.

Potable needs in Illinois are presently met by a combination
of Lake Michigan and groundwater supplies. Under Option 1, some
areas are to be supplied with treated rural stormwater flows and re-

claimed municipal and industrial (M & I) flows. The mechanism for the distribution of potable supplies from sources to need areas is described in Appendix B, Section IV-G.

Under Option 2, withdrawals from Lake Michigan would supply all needs for potable water in the areas presently supplied by withdrawals from Lake Michigan, and all potable deficiencies to the groundwater service areas from Lake Michigan. Reclaimed rural stormwater and M & I flows are not used for any potable reuse needs in this option. The distribution of Lake Michigan water to the potable demand areas is also addressed in Appendix B, Section IV-G.

RECREATIONAL-NAVIGATIONAL REUSE

Recreational flow needs are determined from observations of existing flow regimes in selected streams within the C-SELM area. Supplementary flows are supplied to designated streams through pipes to headwater and downstream supply points to provide a year-round base flow. Criteria for supplementary flow quantity is presented in Appendix B, Section IV-G. Recreational flows are supplied from reclaimed M & I flows.

Navigational requirements are based upon lockage needs. In order to conserve water, all Lake Michigan locks are assumed to be converted to closed systems. Lockage pump-back systems are supplied at lake-lock locations, and thus require no introduction of additional diversion from Lake Michigan.

Recreational-navigational reuse flows are independent of potable reuse flows and are identical for reuse option 1 (3200 cfs restriction) and reuse option 2 (no restriction). Recreational and navigational flows are supplied through a complex distribution system which is described in detail in Appendix B, Section IV-G.

REUSE IMPACT CONSIDERATIONS

The reuse of treated rural, municipal and industrial flows affects the quality of water for recreational and potable uses. These effects depend upon the sequence in which the water is used and then reused. To accomplish an analysis of this pattern of use and reuse, a water movement study, or "water balance", is created, to follow the movement of M & I, stormwater, renovated wastewater and other reuse flows within and between the Illinois and Indiana portions of the C-SELM area.

Recreational water quality affects such important concerns as aquatic community conditions and aesthetic appearance.

One parameter of the quality of the potable water supply which is of particular interest with respect to possible water reuse is the total disssolved solids (TDS) content.

Eash of these points is covered in detail in Appendix B, Section IV-G.

II. SUMMARY OF DESIGN

H. SYNERGISM SYSTEMS

SYNERGISM DEFINED

The dictionary traces the roots of the word synergism to the Greek perfix, syn, meaning with or together and erg (on), meaning work. It is defined as, "the joint action of agents, or drugs, that when taken together increase each other's effectiveness."

The word, as the definition implies, was originally utilized by the medical and chemical professions to describe the action of drugs. However, a more widespread application of the term was introduced by R. Buckminster Fuller, the noted architect and author, to describe structural action.

While retaining the word synergy, Fuller modified the adjective from synergistic to synergetic, from the Greek, syn and ergetikos, meaning energy. This modification implied a cooperative action in energy, strength or endurance. Fuller employed the word, most closely, in relationship with his geodesic dome, which combined the structurally desirable properties of the sphere (dome) and the tetrahedron, thereby vastly increasing its structural strength.

The word synergism is now most commonly understood as meaning an enhancement of any two actions or effects; it is a case of two plus two equalling four, plus. As such, it is a most basic principle in design and development, with items such as high-tensile steel and hybrid corn being prime examples.

In this study, the term synergism is used to describe the cooperative or potentially cooperative utilization of one element by more than one user. This implies that one system or element can, without significant additional cost or development, perform functions in addition to that for which it is designed. This is not meant to imply that the second function has not been considered in the design stage, but merely that its incorporation entailed no, or little, additional cost.

COMMON SYNERGISMS

It would be fair to state that several of the reuse systems

incorporated as basic elements in this wastewater management study can be considered as synergistic. These include the recreational and navigational reuses of the treated waters and the systems designed to recharge potable water supplies. By treating water to NDCP standards, and by distributing these treated waters to augment low flows, numerous and significant recreational possibilities are created.

The beneficial uses of sludge, a by-product of all waste treatment processes, are only recently being recognized. Now, instead of considering sludge as a disposal item, it is being thought of in terms of its nutrient content, its soil-conditioning abilities and its soil-stabilizing abilities. Liquid sludge, from the land treatment, advanced biological or conventional biological processes, can be utilized as a crop fertilizer and as a medium to resculpt and condition strip mine areas. Sludge from the physical-chemical process can be utilized for acidity control and as a soil conditioner.

In addition, by approaching the problem as one of regional wastewater management, a comprehensive approach enables a more efficient analysis, distribution and management of the problem; e.g., upstream and downstream problems are minimized because the broader regional system is being considered and analyzed. Duplication and overlapping can be avoided -- as well as gaps or oversights in the system.

Employed as a reflection or instrument of regional policy, the wastewater management system can be a significant aid in guiding metropolitan growth; can promote balanced land use planning; and can phase construction and operation to maximize efficient use of regional and Federal funds.

UNIQUE SYNERGISMS

In addition to the above common or built-in synergisms, there are several synergisms which are unique to treatment types or system add-ons. These are briefly described, as follows:

Wastewater Irrigation of Crops

Probably the singlemost efficient synergism that can be associated with wastewater management is the utilization of partially treated wastewater as a nutrient and irrigant for crops. In one, simple operation water is stripped of its pollutants, which in turn

serve as crop nutrients. This system also eliminates the need for artificial, chemical fertilizers and the resource and energy consumption required to manufacture them. Simply, it utilizes several elements which would otherwise be disposed of; e.g., wastewater, phosphorus, nitrogen, and eliminates the need for their artificially-produced substitutes. If agreements are made with existing farmers to irrigate their crops on some leasing arrangement, relocations and acquisitions can be minimized and the existing agricultural crops enhanced.

Coordinated Waste Management

Any regional program for wastewater management will permit opportunities for a coordinated program for sludge management. However, the land treatment system not only permits a coordinated program, it supplies the required land area for disposal in the interstitial areas. It further permits a coordinated program for solid waste disposal in the interstitial areas, initially with lagoon over-burden for daily or weekly cover. These coordinated, ancillary functions will have the benefit of an in-place drainage system to prevent leachate migration.

Open Space Acquisition Cost Defrayment

The C-SELM area is an urban region of 7.7 million people, expected to grow to 11.0 million people within the next 25 years. As with any intensively urbanized area, land is at a premium for all uses--but land to serve the recreational-open-space needs of this urban complex is perhaps most critical. The Northeastern Illinois Planning Commission (NIPC) has estimated the open space needs for Northeastern Illinois at 200,000 additional acres by the year 1995; an amount approximating one billion dollars has been estimated as that required for land acquisition over the coming decade.

Because the land treatment system and recreational open space are compatible--if adequate steps are taken to minimize disruption to the natural setting--steps could be undertaken to jointly acquire the necessary land. Two public facilities could thus be facilitated at reduced costs to both. Some compromises would have to be made, however, with odd-shaped, relatively inefficient corridor-area spaces being most desirable to meet recreational demands, and relatively remote, large parcels being most desirable for land treatment. However, shared facilities at intermediate suburban zones are quite feasible, as would be a very large game preserve at a more remote location.

Rural Stormwater/Land Banks

Perhaps the most feasible application of the dual recreational/ treatment use is the typical rural stormwater treatment area. These areas, incorporating storage and settling ponds for watershed areas of an average of 3,000 acres, with spray irrigation on adjacent agricultural land, will serve the stormwater treatment needs of the predominantly rural hinterland of the C-SELM area. However, as the metropolitan area grows, and suburban or urban stormwater treatment facilities are built, portions of these areas may revert to recreational open space with the remaining portions for suburban stormwater storage as described below. These areas will, in effect, act as open space land banks for the growing region.

Suburban Stormwater

Stormwater runoff in medium density suburban areas will be captured in broad, shallow pit areas. These pits may be created specifically as such or be transitions from the rural stormwater areas described above. These pits will be inundated with up to 20 feet of water for substantial periods of time (from 70% of the time in April-May to 10% of the time in September-January). For reasonable periods thereafter, they will contain mucky or residue soils. However, if carefully landscaped and adequately safeguarded, these areas could provide visual open space during the spring and summer and could be used as playlots and sledding-skating areas during the autumn and winter. Alternatively, permanent recreational ponds could be developed at the bottom of these storage areas.

Cooling and Pumped Storage Facilities

As the power demands of the region increase and as the concern for environmental quality grows, methodologies which respect and balance both will be required. The utilization of wastewater storage lagoons as both cooling ponds and pumped storage facilities is a major effort in this direction. This dual use of the storage lagoons for waste heat dissipation in a controlled environment and for power generation during peak hours will aid the power companies in meeting their consumer demands with reduced adverse environmental impacts.

Industrial Incentives

Many of the concerns of the power generating companies are common to a growing number of industries. Their needs for waste-

heat dissipation, exotic waste treatment, and large power consumption could best be satisfied in a land treatment area. Consequently, a land treatment area could well become a development node for large waste-producing or energy-consuming industries. This synergism could reduce industrial pollution while increasing industrial employment opportunities.

Aquaculture

One of the more experimental utilization of wastewater lagoons is that for aquaculture--ranging from algae-production to edible seafoods, such as catfish and shrimp. Recent studies have shown that some shrimp varieties thrive on wastewater nutrients. Just as wastewater can aid crop growth in the soil, so can it aid crop growth in the water itself.

Illustrative sketches of several of these synergisms are shown on Figure B-II-H-1.

SYNERGISMS

WASTEWATER IRRIGATION OF RECREATIONAL OPEN SPACE

SUBURBAN STORMWATER AREAS WITH PART-TIME RECREATIONAL USAGE

ACID STRIP-MINE RECLAMATION WITH SLUDGE

OPEN SPACE CORRIDOR DEVELOPMENT
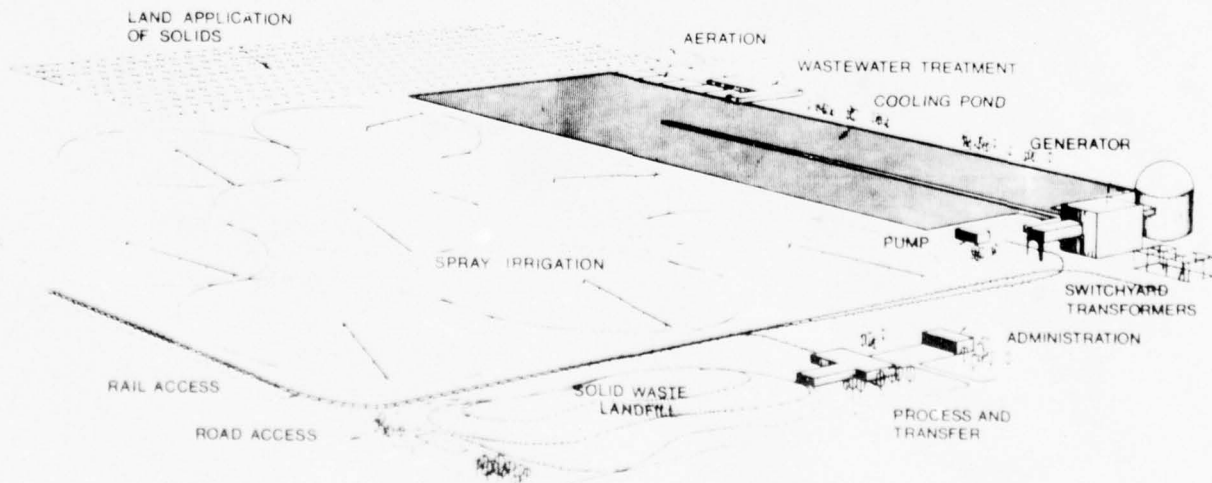
INDUSTRIAL AND RESOURCE DEVELOPMENT OF IRRIGATION SITE

II.   SUMMARY OF DESIGN

I.   NON-STRUCTURAL MANAGEMENT SYSTEMS

DEFINITION

Non-structural management systems are considered to be any of the relatively independent areas of interest in wastewater management that are not specifically dealt with in the construction of wastewater facilities and appurtenances. They are peripheral concerns that sometimes have important implications on effective management, where improvements in present practice may add desirable refinements to the overall program.

Six areas are considered to be of sufficient importance to merit discussion: control of soil erosion, water conservation, septic systems, phosphate detergent bans, control of pleasure and commercial watercraft wastes, and control of water pollution at solid waste landfills.

CONTROL OF SOIL EROSION

The causes and problems related to soil erosion are well documented. Damage from erosion, siltation, pollution, and loss of productivity is costly, unsightly, and a nuisance. Techniques for controlling soil erosion and sedimentation, based on extensive experience, are available for use. Erosion control is practical along some waterways, on major highway construction, sporadically on agricultural land, and in other selected areas. Local ordinances and controls generally are lacking and the need for improvements, possibly by use of a model ordinance, is apparent.

WATER CONSERVATION

Domestic water use is steadily increasing and conservation is worthy of consideration. Water meters and water-saving devices are effective conservation measures, but the benefits to wastewater treatment does not clearly offset the cost of installing them to existing residences. Use in new construction appears to be desirable; further study would be helpful in regard to them and to other means of conservation, such as on-site reuse of wastewater.

SEPTIC SYSTEMS

The widespread and continued use of septic systems warrants the development of a program to deal with existing and new installations, and the development of improved technology and procedures for future systems. Several fertile areas of research and development are suggested in B-IV-I, and a model code is outlined in BA-IV-I that makes use of present knowledge and desirable practice. A program of evaluating existing systems and determining criteria for establishing whether temporary or permanent use should be made of on-site systems is discussed. Central sewer systems are usually considered to be desirable, but the permanent use of on-site systems have definite advantages under certain conditions.

PHOSPHATE DETERGENT BAN

The purpose of reducing phosphorus in wastewater is to improve the quality of flowing streams in the C-SELM area by the reduction in algal growth, the key elements of which are carbon, nitrogen, and phosphorus. The impact of a phosphate detergent ban depends on the relative magnitude of phosphorus that results compared to the carbon and nitrogen present in wastewater. An analysis indicates that the carbon in treatment plant effluent, as presently constituted in the C-SELM area, is the critical nutrient in the production of algae, and that advanced treatment to reduce the level of carbon would be the preferred course of action to achieve intermediate levels of stream eutrophication. It would be necessary to reduce phosphorus much beyond that possible by banning phosphate detergents for baseline or background eutrophic control.

CONTROL OF PLEASURE AND COMMERCIAL WATERCRAFT WASTES

Present onboard and onshore practice for control of this growing source of pollution is discussed in B-IV-I. Regulations of the Environmental Protection Agency are cited as well as other related rules and controls.

CONTROL OF WATER POLLUTION AT SOLID WASTE LANDFILLS

There is a need for improved methods for control of surface water infiltration to landfills and for the prevention of leachate from a landfill polluting the groundwater. The nature of the problem is discussed in Section B-IV-I and improved controls are identified to divert external surface runoff, restrict precipitation from percolating through the refuse, and intercept and treat the remaining groundwater percolation.

TECHNICAL APPENDIX B

FLOW BASIS OF DESIGN

III. FLOW BASIS OF DESIGN

A. PRESENT DOMESTIC-COMMERCIAL AND INDUSTRIAL FLOWS

EXISTING MUNICIPAL TREATMENT FACILITY DATA

The existing (1970-71) operating data for municipal wastewater treatment facilities is presented in the wastewater treatment plant portion of Data Annex B, Section III-A. This inventory of municipal treatment systems includes only those plants whose average daily flow is equal to or greater than 10,000 gallons per day. Miscellaneous treatment systems for schools, motels, restaurants, etc. are not included. The present municipal inventory totals some 132 plants within the C-SELM area.

The wastewater characteristics of the systems vary according to the nature of the waste treated. For example, small suburban systems generally treat wastewater typical of domestic-commercial origin. On the other hand, the three major treatment facilities of the Metropolitan Sanitary District of Greater Chicago (MSD) also treat industrial wastes which comprise some 42% of their total flow. During wet periods, all plants exhibit increasing flows due to stormwater infiltration or combined sewer effects. Except for a limited number of small sewage treatment plants, treatment normally given municipal waste is secondary in nature, typically resulting in an 85-90% reduction in the amount of biochemical oxygen demand (BOD) waste and suspended solids entering the system.

The present estimated municipal wastewater flow generated in the C-SELM study area is approximately 1,600 MGD, of which 86% or 1,370 MGD is contributed by the MSD. The industrial flow portion to the municipal sewer system totals some 600 MGD.

The estimated population serviced by all the municipal systems is approximately 6,965,000 people or 97% of the present population (7,220,000) in the study area.

The waste biological sludges produced by municipal wastewater treatment plants are usually stabilized in anaerobic digestors followed

by lagoon dewatering and subsequent spreading on land for fertilizer and soil conditioner use. Waste biological sludges can also be oxidized, either partially or fully, by wet oxidation or incineration techniques, respectively, with landfilling of the oxidized ash residue. Dry fertilizer of the Milorganite type is made from dried waste activated sludge. The sludge yield from conventional biological waste activated sludge plants is typically some 0.8 tons of dry solids/MG of raw sewage.

EXISTING INDUSTRIAL TREATMENT FACILITY DATA

Presented in the industrial treatment plant section of Data Annex B, Section III-A, is a list of the existing (1970-71) significant industrial wastewater dischargers to surface waters in the C-SELM area. Industrial wastewater treatment operations are considered significant if the total discharge exceeds 5 MGD.

The total significant industrial discharges in the study area generate 12,820 MGD, of which 10,920 MGD or about 85% is utilized for direct cooling purposes; 1,023 MGD or 8% for process wastewater; and 876 MGD or 7% as cooling process wastewater.

The preponderant portion of the industrial flow, 8,135 MGD or 63.5%, is currently discharged into the DesPlaines River Drainage Basin. In general, the surface wastewater industrial discharges in the C-SELM study area can be classified into three main types: steel (2,630 MGD), power (9,660 MGD), and petroleum (220 MGD).

Also presented in Data Annex B, Section III-A, is a list of the waste solids or sludge management practices for the major industries. These generalized management practices are indicative of the present technology being utilized by these industries. It should be noted, however, that the future's increasingly more stringent water quality standards will place greater emphasis on the waste solids management practices of these industries.

III.   FLOW BASIS OF DESIGN

B.   FUTURE DOMESTIC-COMMERCIAL
AND INDUSTRIAL FLOWS

INTRODUCTION

A summary of the results of the municipal and industrial flow projections for the C-SELM design period of 1980-2020 is presented in Table B-III-B-1.   Included in this table are total populations, populations served by public sewers, domestic (including commercial and infiltration) flows, industrial flows and total flows delineated by township and county.   The data portrayed in Figure B-III-B-1 are for the projected C-SELM wastewater service areas for the design years 1990 and 2020.   The average daily domestic flow is projected to increase from the present flow of some 1,000 MGD to over 1,700 MGD in the year 2020.   This increase is attributed to increasing per capita water usage and expanding service areas and populations.   Conversely, the industrial flow projections indicate decreasing industrial flow patterns. As will be discussed in detail in Appendix B, Section IV-B, this decrease in flow results from projected industrial wastewater recycle programs.   The following material in this section describes the procedure used for projecting future C-SELM wastewater flows.

METHODOLOGY FOR FLOW PROJECTIONS

Population projections for each township (including the City of Chicago by sectors) in the C-SELM area were supplied by the Corps of Engineers, Chicago District (hereafter referred to as the Corps). These projections are presented in Table B-III-B-1.   These projections were for the years 1980, 1990, 2000, 2010 and 2020.   Included in the supplied projections were revisions suggested by the Northeastern Illinois Planning Commission (NIPC).   Population data for the period 1950 to 2020 is presented in the Population Projections Section of Data Annex B, Section III-B.

The projections, as supplied, are utilized in composing the total population data in Table B-III-B-1 for 1980, 1990, 2000 and 2020.

C-SELM WASTEWATER FLOWS 1980
BY TOWNSHIPS

Township	Management Watersheds	Total Population (1,000's)				Population Served (1,000's)				Domestic Flow (MGD)				Domestic Flow Class	Out...	
		1980	1990	2000	2020	1980	1990	2000	2020	1980	1990	2000	2020		1980	1990
COOK, ILL.		5,817	6,188	6,426	6,588	5,803	6,188	6,426	6,588	803.1	887.0	958.6	1065.1		19,488.5	24,760
Chicago [a]	4	3,300	3,300	3,325	3,375	3,300	3,300	3,325	3,375	495.0	511.5	532.0	573.8	-A-	11,928.3	14,910
Berwyn [a]	4	185	185	185	185	185	185	185	185	27.8	28.7	29.6	31.5	-A-	880.7	1,122
Bloom	16	117	140	154	165	117	140	154	165	17.6	21.7	24.6	28.1	-A-	494.1	66
Bremen	13,14,16	134	190	226	240	134	190	226	240	15.8	23.9	30.3	36.0	-B-	44.9	6
Calumet	4	26	28	31	35	26	28	31	35	3.1	3.5	4.2	5.3	-B-	117.2	15
Elk Grove	4,5,6	105	112	117	127	105	112	117	127	12.4	14.1	15.7	19.1	-B-	335.9	46
Evanston	4	82	84	86	90	82	84	86	90	12.3	13.0	13.8	15.3	-A-	115.2	14
Lemont	11,12	15	28	40	46	15	28	40	46	1.8	3.5	5.4	6.9	-B-	23.4	3
Leyden [2]	4,5	155	177	192	204	155	177	192	204	18.3	22.3	25.7	30.6	-B-	1,312.4	1,67
Lyons	4,5,10	125	135	143	149	125	135	143	149	14.8	17.0	19.2	22.4	-B-	462.8	60
Maine	3,4,5	178	186	194	201	178	186	194	201	21.0	23.4	26.0	30.2	-B-	326.1	41
New Trier	1,2	66	66	66	66	66	66	66	66	7.8	8.3	8.8	9.9	-B-	39.1	4
Niles	4	130	135	135	135	130	135	135	135	15.4	17.0	18.1	20.3	-B-	1,287.0	1,64
Northfield	2,3	96	117	120	120	96	117	120	120	11.3	14.7	16.1	18.0	-B-	169.9	22
Orland	12,13,14	36	72	88	99	25	72	88	99	3.0	9.1	11.8	14.9	-B-	37.1	5
Palatine	3,6	81	109	133	133	81	109	133	133	9.6	13.7	17.8	20.0	-B-	54.7	8
Palos	11,12	49	57	62	66	49	57	62	66	5.8	7.2	8.3	9.9	-B-	39.1	1
Proviso [c]	4,5,6	205	205	205	205	205	205	205	205	24.2	25.8	27.5	30.8	-A-	595.6	76
Rich	14,16	68	97	116	116	65	97	116	116	7.7	12.2	15.5	17.4	-B-	70.3	10
Shaumburg	6,8	76	106	124	124	76	106	124	124	9.0	13.4	16.6	18.6	-B-	78.1	11
Stickney	4,11	54	64	72	77	54	64	72	77	6.4	8.1	9.6	11.6	-B-	324.8	41
Thornton	4,16	215	250	250	250	215	250	250	250	25.4	31.5	33.5	37.5	-B-	435.5	55
Wheeling	3,4	147	159	166	176	147	159	166	176	17.4	20.0	22.2	26.4	-B-	156.2	22
Worth	4,11,13	172	186	196	204	172	186	196	204	20.3	23.4	26.3	30.6	-B-	160.1	21
DU PAGE, ILL.		706	903	1,084	1,279	675	894	1,084	1,279	79.9	112.7	145.2	192.0		995.2	1,89
Addison	5,6	120	158	183	200	120	158	183	200	14.2	19.0	24.5	30.0	-B-	331.2	60
Bloomingdale	6,7,8	59	86	113	140	45	86	113	140	5.3	10.8	15.1	21.0	-B-	26.6	6
Downers Grove	6,7,10	122	137	144	154	122	137	144	154	14.4	17.3	19.3	23.1	-B-	98.5	18
Lisle	7,8	75	100	126	150	75	100	126	150	8.9	12.6	16.9	22.5	-B-	82.1	16
Milton	7,8	103	150	181	200	103	150	181	200	12.2	18.9	24.3	30.0	-B-	132.3	25
Naperville (part)	8,9	20	35	60	90	15	35	60	90	1.8	4.4	8.0	13.5	-B-	29.5	
Wayne (part)	8	12	20	40	65	9	18	40	65	1.1	2.3	5.4	9.8	-B-	44.7	8
Winfield	8	34	52	72	115	25	45	72	115	3.0	5.7	9.6	17.3	-B-	54.4	11
York	5,6,7	161	165	165	165	161	165	165	165	19.0	20.8	22.1	24.8	-B-	195.9	36
LAKE, ILL.		434	580	693	840	388	540	656	807	49.2	71.5	91.1	123.8		1,194.3	1,7
Antioch (part)	3	5	7	10	12	0	0	0	2	0	0	0	0.3	-B-	5.1	
Avon (part)	3	12	15	22	40	10	15	22	40	1.2	1.9	2.9	6.0	-B-	10.2	
Benton-Zion	1,3	43	53	65	88	35	50	65	88	4.1	6.3	8.7	13.2	-B-	74.2	1
Deerfield-W. Deerfield	1,2	93	125	140	150	93	125	140	150	11.0	15.8	18.8	22.5	-B-	185.5	2
Ela (part)	3	9	13	19	31	3	5	12	25	0.4	0.6	1.6	3.8	-B-	16.6	
Fremont (part)	3	6	9	13	22	3	6	11	22	0.4	0.8	1.5	3.3	-B-	4.4	
Lake Villa (part)	3	7	8	11	22	5	7	9	22	0.6	0.9	1.2	3.3	-B-	1.7	
Libertyville	2,3	40	72	102	129	37	72	102	129	4.4	9.1	13.7	19.4	-B-	140.5	2
Newport	3	5	9	11	20	0	0	2	5	0	0	0.3	0.8	-B-	1.2	
Shields	1,2	68	80	85	85	68	80	85	85	8.0	10.1	11.4	12.8	-B-	140.5	
Vernon	3	22	39	51	60	15	35	48	58	1.8	4.4	6.4	8.7	-B-	20.9	
Warren	3	25	35	44	56	20	30	40	56	2.4	3.7	5.4	8.4	-B-	12.2	
Waukegan	1,2,3	99	115	120	125	99	115	120	125	14.9	17.8	19.2	21.3	-A-	581.9	8
WILL, ILL.		328	466	627	932	261	393	567	896	34.3	53.2	80.1	138.3		846.3	1,7
Channahon	9,15	4	5	6	9	0	0	0	0	0	0	0	0	-B-	37.2	
Crete (part)	16,17	17	22	28	40	10	17	24	37	1.2	2.1	3.2	5.6	-B-	1.4	
DuPage	7,9,10,12	33	48	62	92	21	35	55	92	2.5	4.4	7.4	13.8	-B-	17.3	
Frankfort	14,15	18	32	46	62	11	20	42	60	1.3	2.5	5.6	9.0	-B-	86.5	
Green Garden (part)	14,15	0	1	1	3	0	0	0	0	0	0	0	0	-B-	0.3	
Homer	12,14	15	28	46	90	10	24	41	90	1.2	3.0	5.5	13.5	-B-	6.0	
Jackson (part)	15	2	3	4	8	0	0	0	0	0	0	0	0	-B-	29.8	
Joliet	10,14,15	107	130	157	190	107	130	157	190	16.1	20.2	25.1	32.3	-A-	526.8	
Lockport	9,10,12	40	58	74	104	35	55	74	104	4.1	6.9	9.9	15.6	-B-	108.7	
Manhattan (part)	15	1	2	3	4	0	0	0	0	0	0	0	0	-B-	0.7	
Monee (part)	14,16	35	45	60	85	35	45	60	85	4.1	5.7	8.0	12.8	-B-	9.6	
New Lenox	14,15	16	26	40	70	12	23	37	70	1.4	2.9	5.0	10.5	-B-	4.5	
Plainfield	9	16	24	38	60	10	20	34	60	1.2	2.5	4.6	9.0	-B-	8.9	
Troy	9,10	19	31	45	80	10	24	38	80	1.2	3.0	5.1	12.0	-B-	16.3	
Wheatland	9	5	11	17	35	0	0	5	28	0	0	0.7	4.2	-B-	22.3	
ILLINOIS TOTALS		7,285	8,137	8,830	9,639	7,127	8,015	8,733	9,570	966.5	1124.4	1275.0	1519.2		22,524.3	27,

Table B-III-E-1

M WASTEWATER FLOWS 1980-2020
BY TOWNSHIPS

stic (GD) 2000	2020	Domestic Flow Class	Industrial Output ($ millions)				Industrial Flow (MGD)				Total Domestic & Industrial Flow (MGD)			
			1980	1990	2000	2020	1980	1990	2000	2020	1980	1990	2000	2020
958.6	1065.1		19,488.5	24,764.5	30,480.7	43,043.4	909.5	871.3	881.6	904.7	1712.6	1758.3	1840.2	1969.8
532.0	573.8	-A-	11,928.3	14,916.7	18,100.9	25,255.7	641.9	587.4	584.0	594.0	1136.9	1098.9	1116.0	1167.8
29.6	31.5	-A-	880.7	1,122.0	1,384.4	1,964.8	27.7	29.6	30.8	31.7	55.5	58.3	60.4	63.2
24.6	28.1	-A-	494.1	665.3	866.8	1,230.2	23.4	22.2	23.5	24.6	4..0	43.9	48.1	52.7
30.3	36.0	-B-	44.9	64.5	79.6	113.0	2.9	3.3	3.3	3.3	18.7	27.2	33.6	39.3
4.2	5.3	-B-	117.2	153.9	189.9	269.5	3.7	4.1	4.2	4.4	6.8	7.6	8.4	9.7
15.7	19.1		335.9	464.2	603.4	856.4	10.6	12.3	13.5	13.9	23.0	26.4	29.2	33.0
13.8	15.3	-A-	115.2	146.5	180.7	256.5	3.6	3.9	4.1	4.2	15.9	16.9	17.9	19.5
5.4	6.9	-B-	23.4	34.8	42.9	60.9	12.2	12.5	12.0	11.9	14.0	16.0	17.4	18.8
25.7	30.6	-B-	1,312.4	1,675.6	2,067.4	2,934.2	43.0	45.3	46.9	48.5	61.3	67.6	72.6	79.1
19.2	22.4	-B-	462.8	603.2	759.6	1,078.0	16.3	17.6	18.3	18.9	31.1	34.6	37.5	41.3
26.0	30.2	-B-	326.1	419.5	517.6	734.6	10.8	11.5	11.5	12.3	31.8	34.9	37.5	42.5
8.8	9.9	-B-	39.1	49.7	61.3	86.9	1.2	1.3	1.4	1.4	9.0	9.6	10.2	11.3
18.1	20.3	-B-	1,287.0	1,643.4	2,042.9	2,899.4	42.5	44.6	46.6	48.1	57.8	61.6	64.7	68.4
16.1	18.0	-B-	169.9	223.4	275.7	391.2	5.3	5.9	6.2	6.3	16.6	20.6	22.3	24.3
11.8	14.9	-B-	37.1	54.6	67.4	95.6	1.2	1.4	1.5	1.5	4.2	10.5	13.3	16.4
17.8	20.0	-B-	54.7	84.4	104.1	147.8	1.7	2.2	2.3	2.4	11.3	15.9	20.1	22.4
8.3	9.9	-B-	39.1	57.1	70.4	100.0	1.2	1.5	1.6	1.6	7.0	8.7	9.9	11.5
27.5	30.8	-A-	595.6	762.1	940.3	1,334.5	19.4	20.6	21.4	22.0	43.6	46.4	48.9	52.8
15.5	17.4	-B-	70.3	104.3	128.6	182.6	2.2	2.7	2.8	3.0	9.9	14.9	18.3	20.4
16.6	18.6	-B-	78.1	111.7	183.7	369.5	2.5	2.9	4.1	6.0	11.5	16.3	20.7	24.6
9.6	11.6	-B-	324.8	414.6	511.5	725.9	10.3	11.0	11.4	11.8	16.7	19.1	21.0	23.4
33.5	37.5	-B-	435.5	558.6	689.1	978.1	15.3	15.7	16.1	16.7	40.7	47.2	49.6	54.2
22.2	26.4	-B-	156.2	220.9	318.5	560.8	4.9	5.8	7.1	9.0	22.3	25.8	29.3	35.4
26.3	30.6	-B-	160.1	213.5	294.0	417.3	5.7	6.0	7.0	7.2	26.0	29.4	33.3	37.8
145.2	192.0		995.2	1,895.2	2,678.6	5,149.2	10.6	14.0	17.0	20.1	90.5	126.7	162.2	212.1
24.5	30.0	-B-	331.2	603.0	801.6	1,539.8	3.5	4.5	5.1	6.0	17.7	24.4	29.6	36.0
15.1	21.0	-B-	26.6	66.4	110.6	212.4	0.3	0.5	0.7	0.8	5.6	11.3	15.8	21.8
19.3	23.1	-B-	98.5	185.4	262.6	504.4	1.0	1.4	1.7	2.0	15.4	18.7	21.0	25.1
16.9	22.5	-B-	82.1	160.0	232.2	446.0	0.9	1.2	1.5	1.7	9.8	13.8	18.4	24.2
24.3	30.0	-B-	132.3	255.7	367.6	706.2	1.4	1.9	2.3	2.8	13.6	20.8	26.6	32.8
8.0	13.5	-B-	29.5	60.9	90.7	174.2	0.3	0.4	0.6	0.7	2.1	4.8	8.6	14.2
5.4	9.8	-B-	44.7	87.6	127.8	245.5	0.5	0.6	0.8	1.0	1.6	2.9	6.2	10.8
9.6	17.3	-B-	54.4	113.2	176.9	339.8	0.6	0.8	1.1	1.3	3.6	6.5	10.7	18.6
22.1	24.8	-B-	195.9	363.0	508.6	976.9	2.1	2.7	3.2	3.8	21.1	23.5	25.3	28.6
91.1	123.8		1,194.3	1,776.3	2,474.7	4,744.9	14.4	14.3	15.1	16.6	63.6	85.8	106.2	140.4
0	0.3	-B-	5.1	8.3	11.5	22.1	0	0.1	0.1	0.1	0	0.1	0.1	0.4
2.9	6.0	-B-	10.2	15.9	23.3	44.6	0.1	0.1	0.1	0.1	1.3	2.0	3.0	6.1
8.7	13.2	-B-	74.2	131.1	211.0	431.2	0.4	0.6	0.9	1.2	4.5	6.9	9.6	14.4
18.8	22.5	-B-	185.5	265.8	371.7	712.6	1.0	1.3	1.5	1.9	12.0	17.1	20.3	24.4
1.6	3.8	-B-	16.6	28.1	40.7	78.1	0.1	0.1	0.2	0.2	0.5	0.7	1.8	4.0
1.5	3.3	-B-	4.4	7.9	12.0	23.0	0	0	0	0.1	0.4	0.8	1.5	3.4
1.2	3.3	-B-	1.7	2.4	3.3	6.4	0	0	0	0	0.6	0.9	1.2	3.3
13.7	19.4	-B-	140.5	207.5	290.2	556.5	0.7	1.0	1.2	1.5	5.1	10.1	14.9	20.9
0.3	0.8	-B-	1.2	3.4	7.5	14.3	0	0	0	0	0	0	0.3	0.8
11.4	12.8	-B-	140.5	209.3	292.7	561.2	0.7	1.0	1.2	1.5	8.7	11.1	12.6	14.3
6.4	8.7	-B-	20.9	43.8	71.2	163.0	0.1	0.2	0.3	0.4	1.9	4.6	6.7	9.1
5.4	8.4	-B-	12.2	32.7	87.2	220.4	0.1	0.2	0.4	0.6	2.5	4.0	5.8	9.0
19.2	21.3	-A-	581.9	819.1	1,052.4	1,911.5	11.2	9.7	9.2	9.0	26.1	27.5	28.4	30.3
80.1	138.3		846.3	1,236.5	1,736.1	2,971.6	42.4	29.4	27.1	27.5	76.7	82.6	107.2	165.8
0	0	-B-	37.2	54.5	76.9	132.5	13.0	9.9	8.4	7.0	13.0	9.9	8.4	7.0
3.2	5.6	-B-	1.4	4.2	9.7	23.0	0	0	0.1	0.1	1.2	2.1	3.3	5.7
7.4	13.8	-B-	17.3	33.0	55.9	112.3	0.1	0.2	0.3	0.5	2.6	4.6	7.7	14.3
5.6	9.0	-B-	56.5	96.7	155.2	299.2	0.3	0.5	0.8	1.4	1.6	3.0	6.4	10.4
0	0	-B-	0.3	0.8	1.1	1.9	0	0	0	0	0	0	0	0
5.5	13.5	-B-	6.0	16.6	32.7	72.3	0	0.1	0.2	0.3	1.2	3.1	5.7	13.8
0	0	-B-	29.8	45.2	63.8	109.8	0.1	0.2	0.3	0.5	0.1	0.2	0.3	0.5
25.1	32.3	-A-	526.8	717.5	939.0	1,488.8	13.0	8.1	9.1	11.0	29.1	28.3	34.2	43.3
9.9	15.6	-B-	108.7	155.1	209.8	345.2	15.6	9.9	7.0	4.8	19.7	16.8	16.9	20.4
0	0	-B-	0.7	2.2	3.1	5.3	0	0	0	0	0	0	0	0
8.0	12.8	-B-	9.6	19.6	41.7	95.8	0	0.1	0.2	0.5	4.1	5.8	8.2	13.3
5.0	10.5	-B-	4.5	12.9	27.5	63.4	0	0.1	0.1	0.3	1.4	3.0	5.1	10.8
4.6	9.0	-B-	8.9	15.7	22.1	38.1	0.1	0.1	0.1	0.2	1.3	2.6	4.7	9.2
5.1	12.0	-B-	16.3	26.6	42.2	80.6	0.1	0.1	0.2	0.4	1.3	3.1	5.3	12.4
0.7	4.2	-B-	22.3	35.9	55.4	103.4	0.1	0.1	0.3	0.5	0.1	0.1	1.0	4.7
75.0	1519.2		22,524.3	27,776.3	37,370.1	55,905.1	976.9	929.0	940.8	968.9	1943.4	2053.4	2215.8	2488.1

C-SELM WASTEWATER FLOWS 1980
BY TOWNSHIPS

Township	Management Watersheds	Total Population (1,000's)				Population Served (1,000's)				Domestic Flow (MGD)				Domestic Flow Class		
		1980	1990	2000	2020	1980	1990	2000	2020	1980	1990	2000	2020		1980	1990
LAKE, IND.		577	632	693	815	543	604	672	807	77.8	88.7	101.6	130.7		2,191.9	2,355.?
Calumet	18	220	226	236	256	220	226	236	256	33.0	35.0	37.8	43.5	-A-	687.0	747.9
Center (part)	19	30	35	45	60	25	30	45	60	3.0	3.8	6.0	9.0	-B-	8.8	14.3
Hobart	18, 19	52	64	72	90	52	64	72	90	6.1	8.1	9.6	13.5	-B-	8.8	17.?
North	17-1, 18	206	209	214	226	206	209	214	226	30.9	32.4	34.2	38.4	-A-	1,470.8	1,545.0
Ross	19	41	59	72	100	20	40	60	100	2.4	5.0	8.0	15.0	-B-	5.5	8.?
St. John (part)	17-1, 19	26	38	50	75	20	35	45	75	2.4	4.4	6.0	11.3	-B-	11.0	21.?
Winfield (part)	19	2	3	4	8	0	0	0	0	0	0	0	0	-B-	0	0
LA PORTE, IND.		78	89	99	122	46	57	66	91	6.8	8.4	10.0	14.6		285.8	399.7
Center (part)	22	15	17	20	25	0	0	3	10	0	0	0.4	1.5	-B-	0	0
Cool Spring	20, 21, 22	17	23	28	40	5	15	20	36	0.6	1.9	2.7	5.4	-B-	42.0	56.7
Michigan	21	41	42	43	45	41	42	43	45	6.2	6.5	6.9	7.7	-A-	239.6	331.?
New Durham (part)	20	2	3	3	5	0	0	0	0	0	0	0	0	-B-	0	0
Springfield (part)	21, 22	3	4	5	7	0	0	0	0	0	0	0	0	-B-	4.2	11.?
PORTER, IND.		116	167	235	434	60	107	182	378	7.2	13.6	24.4	56.8		401.4	671.?
Center	20	30	37	46	77	25	33	42	77	3.0	4.2	5.6	11.6	-B-	55.0	85.0
Jackson (part)	20	2	4	7	15	0	0	0	0	0	0	0	0	-B-	0	0
Liberty (part)	20	7	12	20	40	0	2	10	30	0	0.3	1.3	4.5	-B-	3.8	4.?
Pine	20, 21	8	12	17	34	0	2	5	20	0	0.3	0.7	3.0	-B-	2.5	7.?
Portage	18, 20	44	64	90	164	25	50	90	164	3.0	6.3	12.1	24.6	-B-	103.7	225.?
Union (part)	19, 20	3	6	11	24	0	0	0	7	0	0	0	1.1	-B-	0	0
Westchester	20, 21	22	32	44	80	10	20	35	80	1.2	2.5	4.7	12.0	-B-	236.4	349.?
INDIANA TOTALS		771	888	1,027	1,371	649	768	920	1,276	91.8	110.7	136.0	202.1		2,879.1	3,426.?
GRAND TOTALS		8,056	9,025	9,857	11,010	7,776	8,783	9,653	10,846	1,058.3	1,235.1	1,411.0	1,721.3		25,403.4	31,202.?

Notes

a includes Cicero and Oak Park
b includes Norwood Park
c includes River Forest and Riverside (wastewater flows calculated at "B" projection)
d A-City of Chicago, Other Inner City
B-Suburbs

Table B-III-B-1 (Continued)

LM WASTEWATER FLOWS 1980-2020
BY TOWNSHIPS

		Domestic Flow Class	Industrial Output ($ millions)				Industrial Flow (MGD)				Total M & I Flow (MGD)			
000	2020		1980	1990	2020	2020	1980	1990	2000	2020	1980	1990	2000	2020
01.6	130.7		2,191.9	2,355.2	2,546.0	3,030.8	439.2	271.8	207.9	176.1	517.0	360.5	309.5	306.8
37.8	43.5	-A-	687.0	747.9	817.8	997.0	124.6	79.8	70.4	71.8	157.6	114.8	108.2	115.3
6.0	9.0	-B-	8.8	14.3	20.6	33.7	0	0.1	0.1	0.1	3.0	3.9	6.1	9.1
9.6	13.5	-B-	8.8	17.8	25.7	49.1	0	0.1	0.1	0.2	6.1	8.2	9.7	13.7
34.2	38.4	-A-	1,470.8	1,545.6	1,635.6	1,865.1	314.6	191.7	137.1	103.6	345.5	224.1	171.3	142.0
8.0	15.0	-B-	5.5	8.3	12.9	24.5	0	0	0.1	0.1	2.4	5.0	8.1	15.1
6.0	11.3	-B-	11.0	21.3	33.4	61.4	0	0.1	0.1	0.3	2.4	4.5	6.1	13.7
0	0	-B-	0	0	0	0	0	0	0	0	0	0	0	0
10.0	'4.6		285.8	399.7	526.8	829.2	1.6	1.7	2.0	3.2	8.4	10.1	12.0	17.8
0.4	1.5	-B-	0	0	0	0	0	0	0	0	0	0	0.4	1.5
2.7	5.4	-B-	42.0	56.7	71.8	108.4	0.1	0.1	0.1	0.2	0.7	2.0	2.8	5.6
6.9	7.7	-A-	239.6	331.1	431.0	669.8	1.5	1.6	1.9	2.9	7.7	8.1	8.8	10.6
0	0	-B-	0	0	0	0	0	0	0	0	0	0	0	0
0	0	-B-	4.2	11.9	24.0	51.0	0	0	0	0.1	0	0	0	0.1
24.4	56.8		401.4	671.2	996.2	1,809.6	51.9	36.8	39.3	57.2	59.1	50.4	63.7	114.0
5.6	11.6	-B-	55.0	85.0	120.8	207.3	0.1	0.2	0.4	0.7	3.1	4.4	6.0	14.7
0	0	-B-	0	0	0	0	0	0	0	0	0	0	0	0
1.3	4.5	-B-	3.8	4.9	8.3	16.8	0	0	0	0	0	0.3	1.3	4.5
0.7	3.0	-B-	2.5	7.3	12.3	24.3	0	0	0	0.1	0	0.3	0.7	3.1
12.1	24.6	-B-	103.7	225.0	376.8	797.4	12.9	11.5	14.1	24.7	15.9	17.8	26.2	49.3
0	1.1	-B-	0	0	0	0	0	0	0	0	0	0	0	1.1
4.7	12.0	-B-	236.4	349.0	478.0	763.8	38.9	25.1	24.8	31.7	40.1	27.6	29.5	43.7
36.0	202.1		2,879.1	3,426.1	4,069.0	5,669.6	492.7	310.3	249.2	236.5	584.5	421.0	385.2	438.6
111.0	1,721.3		25,403.4	31,202.4	41,439.1	61,574.7	1,469.6	1,239.3	1,190.0	1,205.4	2,527.9	2,474.4	2,601.0	2,926.7

B-III-B-3

VICINITY MAP

SERVICE AREAS IN 1990

1990-2020 SERVICE AREA EXTENSIONS

Figure B-III-B-1

WASTEWATER SERVICE AREAS
1990 AND 2020

SCALE IN MILES

B-III-B-4
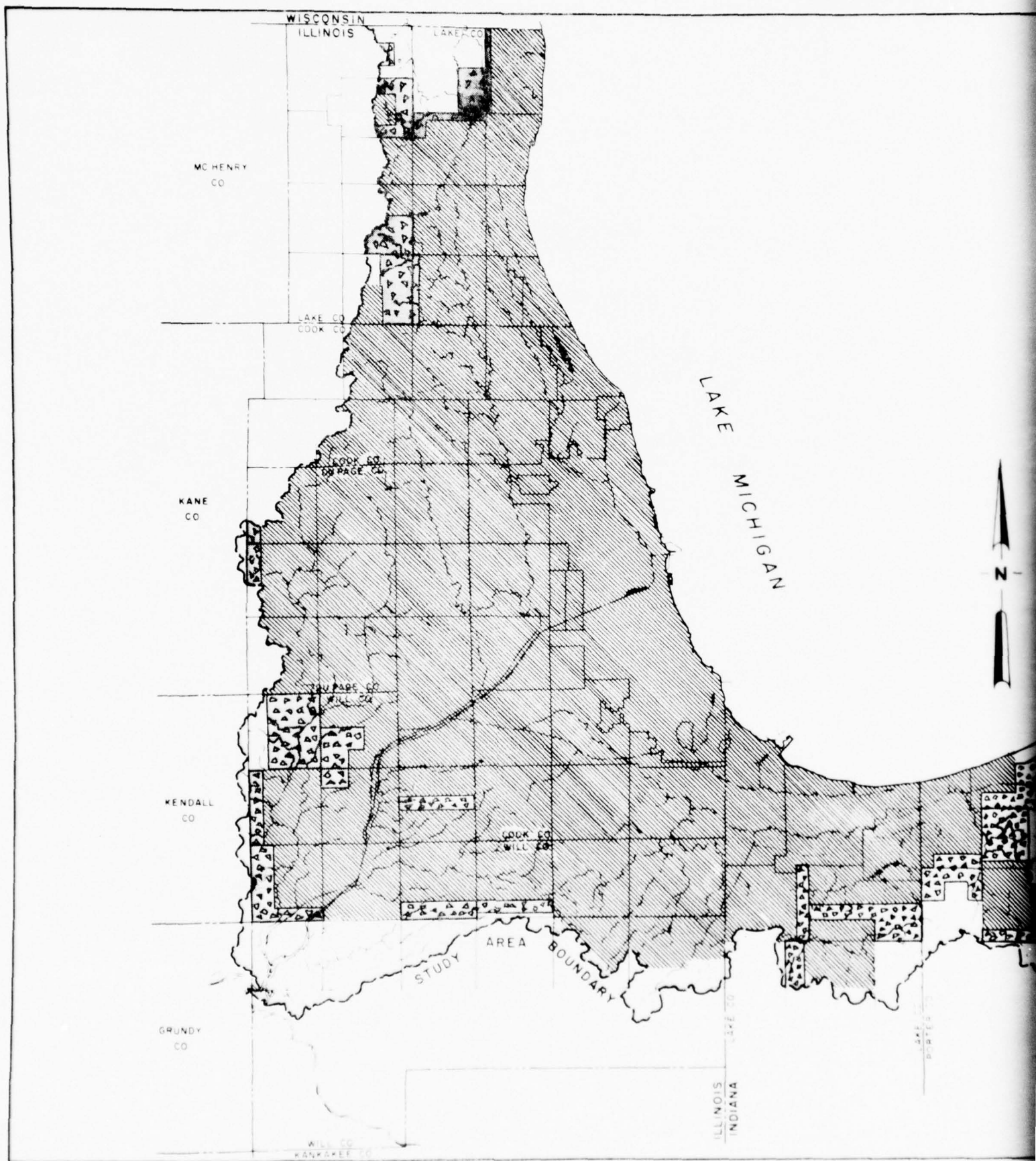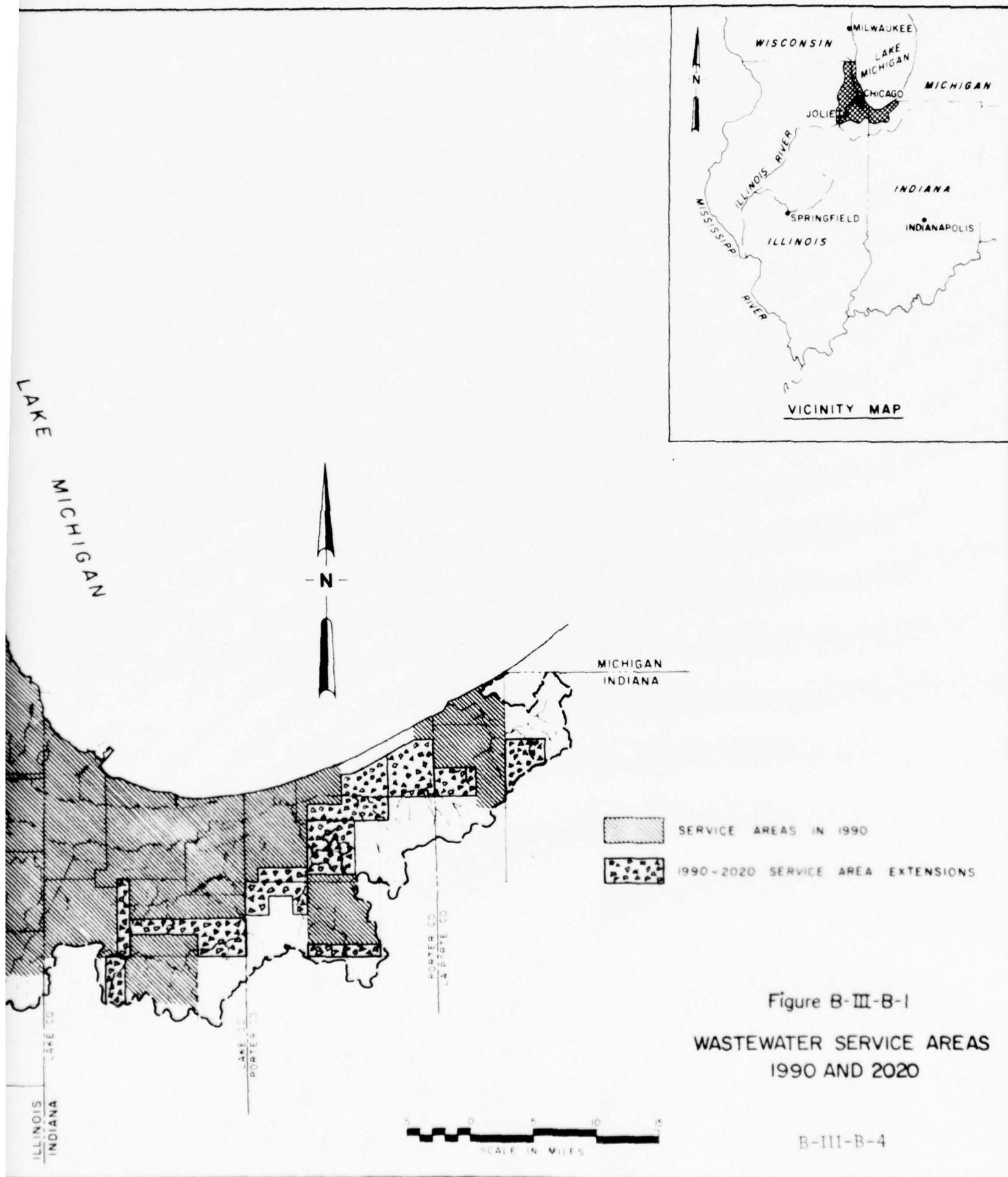
Minor revisions are made to reflect townships not wholly within the C-SELM area (Naperville in DuPage County, for instance).

Using the total populations discussed above, an estimate of population served by public sewers is made for each township in the C-SELM area for the years 1980, 1990, 2000 and 2020 as presented in Table B-III-B-1. The criterion chosen for serving a population is a population density of 2,000 persons/sq. mile or greater as used in the NIPC Wastewater Study [1] and which approximates U. S. Public Health Service "rule of thumb" criteria for environmental planning. [2] Using the township area from the Corps Appendix Table III-A-1 [3] and the total populations from Table B-III-B-1, the densities for each township in 1980, 1990, 2000, 2020 are calculated. Those having a density of 2,000 persons/square mile or greater are assumed to be completely served, i.e., the served population would equal the total township population. For those townships having fewer than 2,000 persons/ square mile an estimate of population served is based on land use and development trends within the particular township and its surrounding area.

Domestic-Commercial Flows

Domestic wastewater flows including commercial and infiltration components are computed for each township in the C-SELM area for the years 1980, 1990, 2000 and 2020 using unit flow projection curves shown in Figure B-III-B-2. The unit flows are expressed in units of gallons/capita/day (gpcd) which, when multiplied by the served populations in Table B-III-B-1, yield the daily domestic flows in gallons/ day which are then converted to million gallons per day (MGD).

In the development of the unit flow projection curves, two different projection situations are assumed based on experience and an evaluation of present available flow data.

Projection A is made for the City of Chicago and other central cities and townships in the C-SELM study area; these include, Evanston, Oak Park, Berwyn, Cicero, Waukegan, Joliet, North (Hammond), Calumet (Gary) and Michigan (Michigan City) townships. This projection recognizes the older development of these areas. The domestic per capita daily consumption is assumed to be increasing at a rate of 0.5 gallons/year. This reflects an increasing utilization of water-using facilities in the urban areas due in large part to the continuous revitalization of these urban areas. The domestic consumption of 95
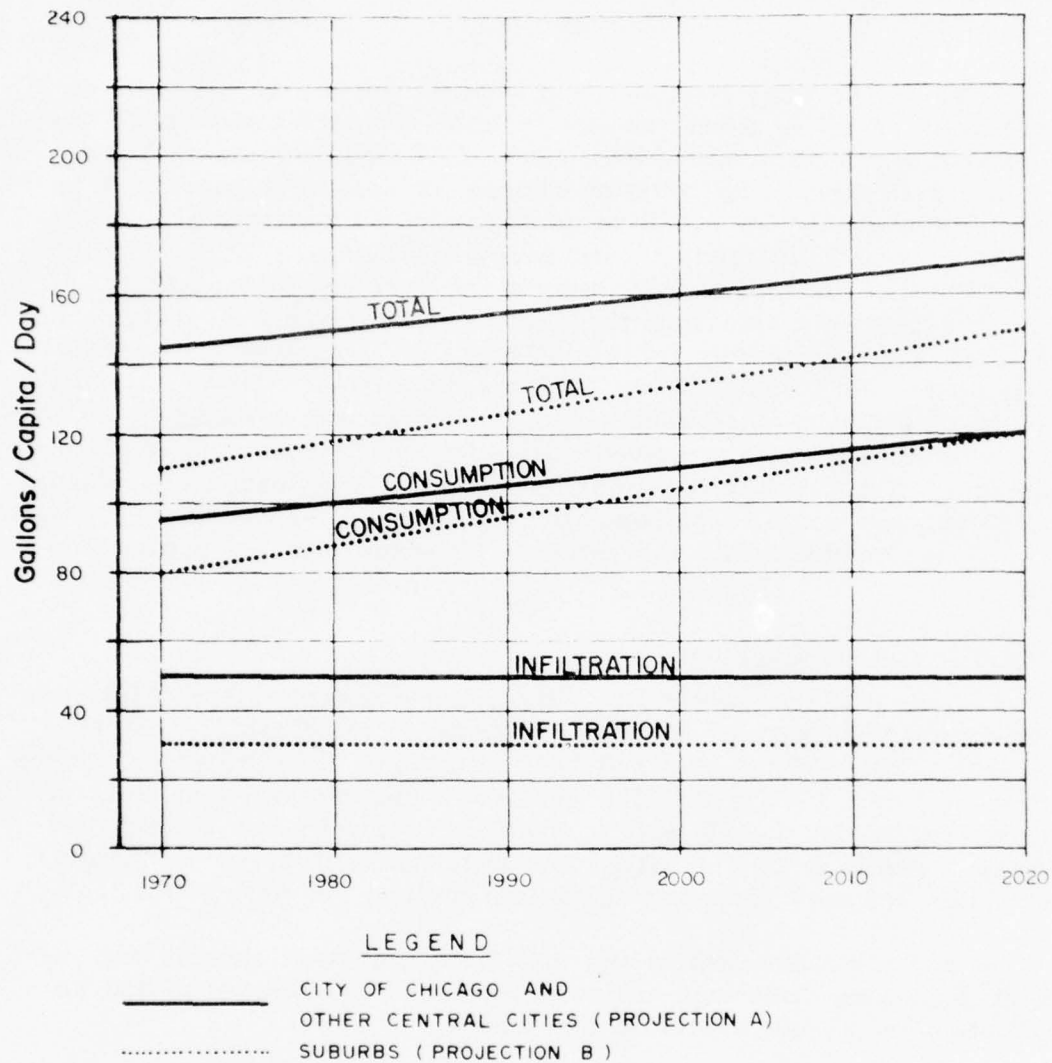
Figure B-III-B-2

C-SELM DOMESTIC UNIT FLOW PROJECTIONS
1970-2020

gpcd for 1970 was determined utilizing data from the Metropolitan Sanitary District of Greater Chicago (MSD) which is in agreement with the municipal sewage treatment plant inventory presented in Data Annex B, Section III-A. 4/ Further data from the MSD indicate that infiltration, including combined sewer discharges to treatment plants in 1970, was approximately 50 gpcd. It is assumed that maintenance of the collector systems for these areas would keep the infiltration constant for the design period.

Projection B is made for the remaining suburban townships in the C-SELM area. The 1970 total domestic consumption is assumed to be 110 gpcd which is representative of these areas as presented in the municipal sewage treatment plant inventory. This unit flow is then broken down into a domestic consumption of 80 gpcd and infiltration of 30 gpcd which is representative of separate sewer systems. The domestic per capita daily consumption is assumed to be increasing at the rate of 0.8 gallons/year reflecting increasing water demands in suburban areas due in large part to increasing commercial growth in these areas. It is also assumed that maintenance of the collector systems will keep the infiltration rate at a constant level. As shown in Figure B-III-B-2, the townships included in Projection B converge toward Projection A over the fifty year design period, reflecting the trend that these areas will, with time, become less distinct from the central city areas.

Specific total unit flows (including consumption and infiltration) derived from the Figure B-III-B-2 curves are shown below for the years 1970, 1980, 1990, 2000 and 2020; these are the unit flows utilized in the domestic flow calculations for the respective years.

Total Unit Flow (gpcd)

Projection	1970	1980	1990	2000	2020
A - City of Chicago and Other Central Cities	145	150	155	160	170
B - Suburbs	110	118	126	134	150

The unit flow curve used for each particular township is indicated in the appropriate column of Table B-III-B-1 by the letter A or B.

Industrial Flows

Introduction. Projecting industrial flows and, implicitly, industrial development within an area as large and complex as the Chicago Metropolitan Area over a long period of time presents many problems not confronted in a typical industrial activity and flow projection endeavor for smaller and less complex areas. Complex shifts in industrial mix and industrial development patterns throughout the metropolitan area are two of the major problems which are confronted. The unusually long period of projection further complicates the task. Consequently, complete reliance on typical extrapolation of present industrial flows is not viewed as the best way to project future industrial flows in the C-SELM area. Therefore, a modified methodology more suited to the vagaries of projecting industrial activity in the complex C-SELM area is developed.

The most complete available data on future industrial activity are net product-value added forecasts for each industry present in the metropolitan area as developed by the Corps.[3/] A critical part of an industrial flow projection task is the necessity to identify industrial activity for those industries which generate proportionally large wastewater flows. Since the Corps value-added projections are broken down by SIC types, the identification of output activity for these large wastewater generating industries can be readily facilitated. For these reasons, namely the comprehensiveness of the Corps value-added data and the ability to separate that portion of value-added contributed by high wastewater generating industries, the Corps value-added projections are utilized as the basis upon which to project future industrial flows.

Several modifications and operations are made upon the Corps value-added projection data. The derivation of industrial flows for each township in 1980, 1990, 2000 and 2020 entails three major steps as follows:

1. An estimation of value-added is made by townships including that component of value-added contributed by each of several specified critical industries (i.e., steel and petroleum).

2. Derivation of unit flow factors is made and verified independently for each of two critical industries with

units of gallons/day/dollar value-added from the Corps data.[3/] Determination is also made of the per capita daily industrial flow to municipal wastewater treatment systems based on present conditions for each county. Projection of future industrial flows to municipal systems is based on the population served by municipal systems with subsequent allocation of county total to townships according to proportion of value-added in each township. An allocation of non-critical surface flows to respective townships is also provided.

3.   Multiplication of unit flow factors by respective components of critical industry value-added projections is made to obtain critical industry waste flow. Total industrial flow then becomes the sum, for each township, of critical industry flow, industrial flow to municipal sewers and noncritical surface flow. These industrial flows are presented in detail in Table B-III-B-2.

These three steps are now elaborated upon in turn.

Estimation of value-added. The Corps provided projections of value-added for each county for all manufacturing industries (SIC groups 20 to 39, inclusive). These are summed for all SIC two-digit groups (i.e., 20 to 39, inclusive) to obtain total value-added for manufacturing by county for the years 1980, 1990, 2000 and 2020.

It is felt that the Corps value-added projections by township did not adequately reflect land use and development trends within the metropolitan area because of the nature in which these township projections were made. Therefore, a supplemental means for allocating value-added county totals among townships is developed.

NIPC employment projections through 1995 are considered for use in the allocations because these projections are based on an opportunity model encompassing land use, spatial, and industrial development trends.[5/] It is felt that the NIPC employment projections adequately reflect the spatial distribution of industrial activity. Consequently, the NIPC projections are used to disaggregate Corps value added figures from the county level to the township level. 1980 and 1990 employment by township are obtained by straight line interpolation of the 1975, 1985 and 1995 NIPC figures. Proportions of total

B-III-B-9

INDUSTRIAL FLOW CALCULATIO

	1980					1990					Ind. Flow... Municipal... (MGD)
	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. SIC 291 (MGD)	to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. SIC 291 (MGD)	to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	
COOK COUNTY											
Township											
Chicago	375.2	93.9	104.9	67.9	641.9	393.9	88.3	59.9	45.3	587.4	403.2
Berwyn	27.7	0	0	0	27.7	29.6	0	0	0	29.6	30.8
Bloom	15.6	0	7.8	0	23.4	17.6	0	4.6	0	22.2	19.3
Bremen	1.4	1.4	0.1	0	2.9	1.7	1.5	0.1	0	3.3	1.6
Calumet	3.7	0	0	0	3.7	4.1	0	0	0	4.1	4.2
Elk Grove	10.5	0	0.1	0	10.6	12.2	0	0.1	0	12.3	13.4
Evanston	3.6	0	0	0	3.6	3.9	0	0	0	3.9	4.1
Lemont	0.7	11.5	0	0	12.2	0.9	11.6	0	0	12.5	1.0
Leyden	41.3	0	1.7	0	43.0	44.3	0	1.0	0	45.3	46.0
Lyons	14.5	0.9	0.9	0	16.3	16.0	1.0	0.6	0	17.6	16.9
Maine	10.2	0	0.6	0	10.8	11.1	0	0.4	0	11.5	11.5
New Trier	1.2	0	0	0	1.2	1.3	0	0	0	1.3	1.4
Niles	40.5	0	2.0	0	42.5	43.4	0	1.2	0	44.6	45.5
Northfield	5.3	0	0	0	5.3	5.9	0	0	0	5.9	6.2
Orland	1.2	0	0	0	1.2	1.4	0	0	0	1.4	1.5
Palatine	1.7	0	0	0	1.7	2.2	0	0	0	2.2	2.3
Palos	1.2	0	0	0	1.2	1.5	0	0	0	1.5	1.6
Proviso	18.8	0	0.6	0	19.4	20.2	0	0.4	0	20.6	21.0
Rich	2.2	0	0	0	2.2	2.7	0	0	0	2.7	2.8
Shaumburg	2.5	0	0	0	2.5	2.9	0	0	0	2.9	4.1
Stickney	10.3	0	0	0	10.3	11.0	0	0	0	11.0	11.4
Thornton	13.8	0	1.5	0	15.3	14.8	0	0.9	0	15.7	15.3
Wheeling	4.9	0	0	0	4.9	5.8	0	0	0	5.8	7.1
Worth	5.0	0	0.7	0	5.7	5.6	0	0.4	0	6.0	6.6
TOTAL	613.0	107.7	120.9	67.9	909.5	654.0	102.4	69.6	45.3	871.3	679.0
DU PAGE COUNTY											
Township											
Addison	3.5	0	0	0	3.5	4.5	0	0	0	4.5	5.1
Bloomingdale	0.3	0	0	0	0.3	0.5	0	0	0	0.5	0.7
Downers Grove	1.0	0	0	0	1.0	1.4	0	0	0	1.4	1.7
Lisle	0.9	0	0	0	0.9	1.2	0	0	0	1.2	1.5
Milton	1.4	0	0	0	1.4	1.9	0	0	0	1.9	2.3
Naperville (P)	0.3	0	0	0	0.3	0.4	0	0	0	0.4	0.6
Wayne (P)	0.5	0	0	0	0.5	0.6	0	0	0	0.6	0.8
Winfield	0.6	0	0	0	0.6	0.8	0	0	0	0.8	1.1
York	2.1	0	0	0	2.1	2.7	0	0	0	2.7	3.2
TOTAL	10.6	0	0	0	10.6	14.0	0	0	0	14.0	17.0
LAKE COUNTY											
Township											
Antioch	0	0	0	0	0	0.1	0	0	0	0.1	0.
Avon	0.1	0	0	0	0.1	0.1	0	0	0	0.1	0.
Benton-Zion	0.4	0	0	0	0.4	0.6	0	0	0	0.6	0.
Deerfield	1.0	0	0	0	1.0	1.3	0	0	0	1.3	1.
Ela	0.1	0	0	0	0.1	0.1	0	0	0	0.1	0.
Fremont	0	0	0	0	0	0	0	0	0	0	0
Lake Villa	0	0	0	0	0	0	0	0	0	0	0
Libertyville	0.7	0	0	0	0.7	1.0	0	0	0	1.0	1.
Newport	0	0	0	0	0	0	0	0	0	0	0
Shields	0.7	0	0	0	0.7	1.0	0	0	0	1.0	1.
Vernon	0.1	0	0	0	0.1	0.2	0	0	0	0.2	0.
Warren	0.1	0	0	0	0.1	0.2	0	0	0	0.2	0.
Waukegan	2.9	0	1.9	6.4	11.2	3.9	0	1.0	4.8	9.7	4.
TOTAL	6.1	0	1.9	6.4	14.4	8.5	0	1.0	4.8	14.3	10.
WILL COUNTY											
Township											
Channahon	0.2	0	0	12.8	13.0	0.3	0	0	9.6	9.9	0.
Crete	0	0	0	0	0	0	0	0	0	0	0.
DuPage	0.1	0	0	0	0.1	0.2	0	0	0	0.2	0.
Frankfort	0.3	0	0	0	0.3	0.5	0	0	0	0.5	0
Green Garden	0	0	0	0	0	0	0	0	0	0	
Homer	0	0	0	0	0	0.1	0	0	0	0.1	0
Jackson	0.1	0	0	0	0.1	0.2	0	0	0	0.2	0
Joliet	2.6	0	10.4	0	13.0	3.6	0	4.5	0	8.1	4
Lockport	0.5	15.1	0	0	15.6	0.8	9.1	0	0	9.9	1
Manhattan	0	0	0	0	0	0	0	0	0	0	
Monee	0	0	0	0	0	0.1	0	0	0	0.1	0
New Lenox	0	0	0	0	0	0.1	0	0	0	0.1	0
Plainfield	0.1	0	0	0	0.1	0.1	0	0	0	0.1	0
Troy	0.1	0	0	0	0.1	0.1	0	0	0	0.1	0
Wheatland	0.1	0	0	0	0.1	0.1	0	0	0	0.1	0
TOTAL	4.1	15.1	10.4	12.8	42.4	6.2	9.1	4.5	9.6	29.4	8

Table B-III-B-2

INDUSTRIAL FLOW CALCULATIONS

1990			2000					2020				
to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. to Surface SIC 291 (MGD)	to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. to Surface SIC 291 (MGD)	to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)
59.9	45.3	587.4	403.2	88.5	54.6	37.7	584.0	408.3	93.8	61.0	30.9	594.0
0	0	29.6	30.8	0	0	0	30.8	31.7	0	0	0	31.7
4.6	0	22.2	19.3	0	4.2	0	23.5	19.9	0	4.7	0	24.6
0.1	0	3.3	1.8	1.4	0.1	0	3.3	1.8	1.4	0.1	0	3.3
0	0	4.1	4.2	0	0	0	4.2	4.4	0	0	0	4.4
0.1	0	12.3	13.4	0	0.1	0	13.5	13.8	0	0.1	0	13.9
0	0	3.9	4.1	0	0	0	4.1	4.2	0	0	0	4.2
0	0	12.5	1.0	11.0	0	0	12.0	1.0	10.9	0	0	11.9
1.0	0	45.3	46.0	0	0.9	0	46.9	47.5	0	1.0	0	48.5
0.6	0	17.6	16.9	0.9	0.5	0	18.3	17.4	0.9	0.6	0	18.9
0.4	0	11.5	11.5	0	0	0	11.5	11.9	0	0.4	0	12.3
0	0	1.3	1.4	0	0	0	1.4	1.4	0	0	0	1.4
1.2	0	44.6	45.5	0	1.1	0	46.6	46.9	0	1.2	0	48.1
0	0	5.9	6.2	0	0	0	6.2	6.3	0	0	0	6.3
0	0	1.4	1.5	0	0	0	1.5	1.5	0	0	0	1.5
0	0	2.2	2.3	0	0	0	2.3	2.4	0	0	0	2.4
0	0	1.5	1.6	0	0	0	1.6	1.6	0	0	0	1.6
0.4	0	20.6	21.0	0	0.4	0	21.4	21.6	0	0.4	0	22.0
0	0	2.7	2.8	0	0	0	2.8	3.0	0	0	0	3.0
0	0	2.9	4.1	0	0	0	4.1	6.0	0	0	0	6.0
0	0	11.0	11.4	0	0	0	11.4	11.8	0	0	0	11.8
0.9	0	15.7	15.3	0	0.8	0	16.1	15.8	0	0.9	0	16.7
0	0	5.8	7.1	0	0	0	7.1	9.0	0	0	0	9.0
0.4	0	6.0	6.6	0	0.4	0	7.0	6.8	0	0.4	0	7.2
69.6	**45.3**	**871.3**	**679.0**	**101.8**	**63.1**	**37.7**	**881.6**	**696.0**	**107.0**	**70.8**	**30.9**	**904.7**
0	0	4.5	5.1	0	0	0	5.1	6.0	0	0	0	6.0
0	0	0.5	0.7	0	0	0	0.7	0.8	0	0	0	0.8
0	0	1.4	1.7	0	0	0	1.7	2.0	0	0	0	2.0
0	0	1.2	1.5	0	0	0	1.5	1.7	0	0	0	1.7
0	0	1.9	2.3	0	0	0	2.3	2.8	0	0	0	2.8
0	0	0.4	0.6	0	0	0	0.6	0.7	0	0	0	0.7
0	0	0.6	0.8	0	0	0	0.8	1.0	0	0	0	1.0
0	0	0.8	1.1	0	0	0	1.1	1.3	0	0	0	1.3
0	0	2.7	3.2	0	0	0	3.2	3.8	0	0	0	3.8
0	**0**	**14.0**	**17.0**	**0**	**0**	**0**	**17.0**	**20.1**	**0**	**0**	**0**	**20.1**
0	0	0.1	0.1	0	0	0	0.1	0.1	0	0	0	0.1
0	0	0.1	0.1	0	0	0	0.1	0.1	0	0	0	0.1
0	0	0.6	0.9	0	0	0	0.9	1.2	0	0	0	1.2
0	0	1.3	1.5	0	0	0	1.5	1.9	0	0	0	1.9
0	0	0.1	0.2	0	0	0	0.2	0.2	0	0	0	0.2
0	0	0	0	0	0	0	0	0.1	0	0	0	0.1
0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	1.0	1.2	0	0	0	1.2	1.5	0	0	0	1.5
0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	1.0	1.2	0	0	0	1.2	1.5	0	0	0	1.5
0	0	0.2	0.3	0	0	0	0.3	0.4	0	0	0	0.4
0	0	0.2	0.4	0	0	0	0.4	0.6	0	0	0	0.6
1.0	4.8	9.7	4.4	0	0.8	4.0	9.2	5.1	0	0.7	3.2	9.0
1.0	**4.8**	**14.3**	**10.3**	**0**	**0.8**	**4.0**	**15.1**	**12.7**	**0**	**0.7**	**3.2**	**16.6**
0	9.6	9.9	0.4	0	0	8.0	8.4	0.6	0	0	6.4	7.0
0	0	0	0.1	0	0	0	0.1	0.1	0	0	0	0.1
0	0	0.2	0.3	0	0	0	0.3	0.5	0	0	0	0.5
0	0	0.5	0.8	0	0	0	0.8	1.4	0	0	0	1.4
0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0.1	0.2	0	0	0	0.2	0.3	0	0	0	0.3
0	0	0.2	0.3	0	0	0	0.3	0.5	0	0	0	0.5
4.5	0	8.1	4.8	0	4.3	0	9.1	7.2	0	3.8	0	11.0
0	0	9.9	1.1	5.9	0	0	7.0	1.6	3.2	0	0	4.8
0	0	0	0	0	0	0	0	0	0	0	0	0
0	0	0.1	0.2	0	0	0	0.2	0.5	0	0	0	0.5
0	0	0.1	0.1	0	0	0	0.1	0.3	0	0	0	0.3
0	0	0.1	0.1	0	0	0	0.1	0.2	0	0	0	0.2
0	0	0.1	0.2	0	0	0	0.2	0.4	0	0	0	0.4
0	0	0.1	0.3	0	0	0	0.3	0.5	0	0	0	0.5
4.5	**9.6**	**29.4**	**8.9**	**5.9**	**4.3**	**8.0**	**27.1**	**14.1**	**3.2**	**3.8**	**6.4**	**27.5**

	1980					1990					Ind.
	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. SIC 291 (MGD)	to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. SIC 291 (MGD)	to Surface SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	Munici (MG
LAKE, INDIANA											
Township											
Calumet	2.7	0	81.9	40.0	124.6	3.0	0	46.8	30.0	79.8	
Center	0	0	0	0	0	0.1	0	0	0	0.1	
Hobart	0	0	0	0	0	0.1	0	0	0	0.1	
North	5.8	220.5	69.3	19.0	314.6	6.2	126.0	45.2	14.3	191.7	
Ross	0	0	0	0	0	0	0	0	0	0	
St. Johns	0	0	0	0	0	0.1	0	0	0	0.1	
Winfield	0	0	0	0	0	0	0	0	0	0	
TOTAL	8.5	220.5	151.2	59.0	439.2	9.5	126.0	92.0	44.3	271.8	
PORTER, INDIANA											
Township											
Center	0.1	0	0	0	0.1	0.2	0	0	0	0.2	
Jackson	0	0	0	0	0	0	0	0	0	0	
Liberty	0	0	0	0	0	0	0	0	0	0	
Pine	0	0	0	0	0	0	0	0	0	0	
Portage	0.2	0	12.7	0	12.9	0.6	0	10.9	0	11.5	
Union	0	0	0	0	0	0	0	0	0	0	
Westchester	0.5	0	38.4	0	38.9	0.9	0	24.2	0	25.1	
TOTAL	0.8	0	51.1	0	51.9	1.7	0	35.1	0	36.8	
LA PORTE, INDIANA											
Township											
Center	0	0	0	0	0	0	0	0	0	0	
Cool Spring	0.1	0	0	0	0.1	0.1	0	0	0	0.1	
Michigan	0.6	0	0.9	0	1.5	0.8	0	0.8	0	1.6	
New Durham	0	0	0	0	0	0	0	0	0	0	
Springfield	0	0	0	0	0	0	0	0	0	0	
TOTAL	0.7	0	0.9	0	1.6	0.9	0	0.8	0	1.7	

Table B-III-B-2 (Continued)

...STRIAL FLOW CALCULATIONS

...urface 331 (.D)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	2000					2020				
			Ind. Flow to Municipal Sewers (MGD)	Critical Ind. to Surface SIC 291 (MGD)	SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)	Ind. Flow to Municipal Sewers (MGD)	Critical Ind. to Surface SIC 291 (MGD)	SIC 331 (MGD)	Non-Critical to Surface (MGD)	Total Ind. Flow (MGD)
	30.0	79.8	3.4	0	42.0	25.0	70.4	4.2	0	47.6	20.0	71.8
	0	0.1	0.1	0	0	0	0.1	0.1	0	0	0	0.1
	0	0.1	0.1	0	0	0	0.1	0.2	0	0	0	0.2
.2	14.3	191.7	6.8	77.7	40.7	11.9	137.1	7.8	39.9	46.4	9.5	103.6
	0	0	0.1	0	0	0	0.1	0.1	0	0	0	0.1
	0	0.1	0.1	0	0	0	0.1	0.3	0	0	0	0.3
	0	0	0	0	0	0	0	0	0	0	0	0
.0	44.3	271.8	10.6	77.7	82.7	36.9	207.9	12.7	39.9	94.0	29.5	176.1
	0	0.2	0.4	0	0	0	0.4	0.7	0	0	0	0.7
	0	0	0	0	0	0	0	0	0	0	0	0
	0	0	0	0	0	0	0	0	0	0	0	0
.9	0	0	0	0	0	0	0	0.1	0	0	0	0.1
.9	0	11.5	1.1	0	13.0	0	14.1	2.6	0	22.1	0	24.7
.2	0	0	0	0	0	0	0	0	0	0	0	0
.2	0	25.1	1.4	0	23.4	0	24.8	2.5	0	29.2	0	31.7
.1	0	36.8	2.9	0	36.4	0	39.3	5.9	0	51.3	0	57.2
	0	0	0	0	0	0	0	0	0	0	0	0
	0	0.1	0.1	0	0	0	0.1	0.2	0	0	0	0.2
.8	0	1.6	0.9	0	1.0	0	1.9	1.1	0	1.8	0	2.9
	0	0	0	0	0	0	0	0	0	0	0	0
	0	0	0	0	0	0	0	0.1	0	0	0	0.1
.8	0	1.7	1.0	0	1.0	0	2.0	1.4	0	1.8	0	3.2

B-III-B-11

county manufacturing employment for each township are then derived from the 1980 and 1990 figures. Proportions for 2000, and 2020 are estimated from the 1980-1990 base projections taking cognizance of likely industrial growth patterns beyond 1990. A matrix of township employment as a proportion of total county employment is thus derived for all Illinois townships for the years 1980, 1990, 2000 and 2020.

For Indiana, the Lake-Porter Planning Commission has data similar in format to that of NIPC regarding manufacturing employment by township to 1995.[6/] These employment projections are interpolated to 1980 and 1990 in the same manner as for Illinois and proportions of the county total employment are similarly derived. These proportions are projected to 2000 and 2020 in a manner which reflected growth and development trends in Lake and Porter Counties. The county totals for value-added are then allocated among townships in the same manner as was done for the Illinois counties and townships. No employment projections are available for LaPorte County. Therefore, employment is projected based on extrapolations of past employment. The said projected employment is then distributed among the three townships based on existing employment.

Thus, a matrix of value-added for each township in the C-SELM area for the years 1980, 1990, 2000 and 2020 is developed and presented in Table B-III-B-1.

Unit flow factors for critical industries. In reviewing present industrial flows, it is found that several key industries generate quite large wastewater flows. These industries are identified as the following:

SIC 3-digit Classification	Industry
291	Petroleum Refineries
331	Steel

In addition, several other industries are found to be presently generating large flows; among them are: Union Carbide, the Joliet Munitions Plant, and Abbott Laboratories. The projection and allocation of these flows among townships are subsequently discussed.

In the case of the critical industries, petroleum and steel, it is necessary to determine the component of value-added at the township level attributable to them. This is done by taking the Corps township value-added projections for all industries and identifying the value-added figures from SIC groups corresponding to these critical industries for the years 1980, 1990, 2000 and 2020. These critical industry value-added figures are subsequently used in the calculation of critical industry flows at the township level by multiplying them by the time-adjusted flow factor of these industries as derived below.

Industrial unit flow factors expressed in units of gallons/day/dollar value-added are presented in the Corps Appendix, Table III-A-2 for 1970.[3/] These flow factors for SIC categories 291 and 331 are verified independently and are found to be in agreement with the Corps factors of 1.400 and 0.970, respectively.

Unit flows within these two industries are envisioned to be reduced over the fifty-year period to reflect the effect of recycling process and cooling waters. The reduced flow in later years is shown as a proportion of the 1970 unit flow in the table below. The derivation of these reduction factors are detailed in Appendix B, Section IV-B.

Critical Industry	Proportion of 1970 Unit Flow				
Classification	1970	1980	1990	2000	2020
291	1.00	.450	.250	.150	.075
331	1.00	.200	.100	.080	.075

The time adjusted flow factors for the critical industries thus become:

Critical Industry	Flow Factor (gallons/day/dollar output)				
SIC Classification	1970	1980	1990	2000	2020
291	1.400	.630	.350	.210	.105
331	0.970	.192	.097	.078	.073

Unit flow factors for noncritical industries. Approximately 350 MGD of wastewater not from the critical industries was discharged to surface streams in 1970. The dispersion of these industries among SIC categories does not render this group readily amenable to the flow projection methodology used for the critical industries. Therefore, a different, more workable, procedure is used.

A recycling effort within these industries is estimated to result in the reduction of the 350 MGD flow to that proportion shown in the following graph. Thus, by 2020, the flow from this group of industries would be (0.20) (350 MGD) = 70 MGD.
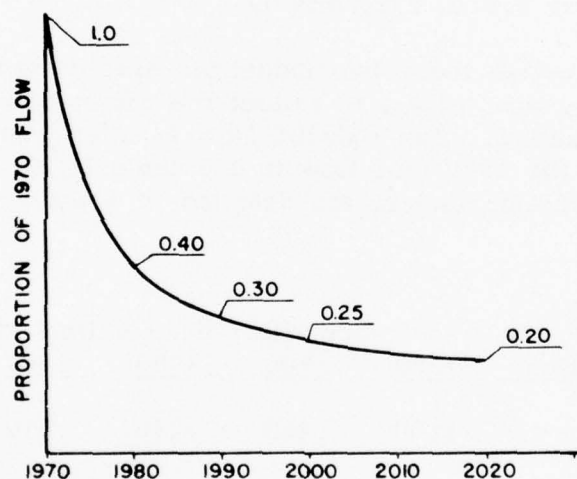
The total flow is allocated among those townships from which it is generated in proportion to the 1970 flow in the given township, i.e., in 2020 approximately 70 MGD would be the control total to be allocated among those townships in which this type of industry is located. Flows so allocated are shown in Table B-III-B-2 under the heading "Noncritical to Surface (MGD)."

Industrial flows presently discharging to municipal systems.
This component of industrial flow is attributable to noncritical indus-
tries which presently discharge to municipal sewers and therefore
tend to have relatively low flows. Present experience in the
C-SELM area shows this flow to be in the range of 0-50% of the total
municipal treatment plant flow. Based on the municipal sewage treat-
ment plant inventory presented in Data Annex B, Section III-A, a rep-
resentative proportion of the total municipal flow by county is assumed
to be industrial. By dividing this industrial flow to municipal systems
by the present population served, an industrial/capita unit flow is
obtained. Assuming that industrial flows to municipal systems will be
proportional to the population served, industrial flows to municipal
systems for the period 1980-2020 (this does not include industrial
flows that are presently discharging to surface waters), for each county,
can be calculated by multiplying the industrial/capita unit flow by the
projected population served. Subsequent to the derivation of the county
totals, an allocation of these totals among townships is made accord-
ing to the proportion of value-added within each township (i.e., a
township having 10% of the county's value added would be allocated
10% of the county's industrial flow to municipal treatment systems).

The specific proportions used for calculating the county total,
together with the industrial/capita unit flow, are shown below. These
factors are assumed to remain constant over the 1970-2020 period.

County	Present Industrial Flow/Total Municipal STP Flow (%)	Industrial Per Capita Unit Flow (gpcd)
Cook	42.5	105.6
DuPage	10	15.7
Lake, Ill.	10	15.7
Will	10	15.7
Lake, Ind.	10	15.7
Porter	10	15.7
LaPorte	10	15.7

The flows allocated as described above are shown in Table B-III-B-2.

Summary of industrial flows. Industrial flows to sewers are derived as described above. Critical industry flows are the product of the township value-added for the particular critical industry and the time-dependent unit flow factor. The noncritical surface discharge flows are derived as discussed earlier. The total industrial flow is the sum of the three components (as applicable to a given township). Total industrial flows for townships by decade are shown in Table B-III-B-1. Also presented in this table are the total domestic-commercial and industrial projected flows by townships for the years 1980, 1990, 2000 and 2020.

Power Industry Flows

Cooling water flows associated with power generation from either fossil fuel-fired or nuclear plants have been excluded from the flow projections developed in this section. The power producing utilities can be expected to meet the evolving thermal standards for discharges to watercourses as they are promulgated by the governmental agencies. Heat dissipation by evaporation appears to be the technological answer to thermal discharge concerns. Therefore, the electric utility can be external to the regional management systems and still be consistent with the ultimate water quality goal for the region.

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III. FLOW BASIS OF DESIGN

C. PRESENT AND FUTURE STORMWATER FLOWS

LAND USE CLASSIFICATION

Stormwater flow projections are based on runoff studies and land development projections. This takes into account the fact that annual stormwater runoff is greatest in the more impervious, highly developed urban areas as contrasted with the more open, undeveloped rural areas.

The C-SELM region is divided into 22 watersheds as shown in Figure B-III-C-1. The land development of each watershed is designated as one of the following three categories according to the population density and manufacturing employment density:

1. Urban Area - The average population density is 10,000 persons per square mile. The range of urban land use is for population densities greater than 5,000 persons per square mile and/or a dense manufacturing employment density.

2. Suburban Area - The average population density is 4,000 persons per square mile. The suburban area land use ranges from population densities between 2,000 and 5,000 persons per square mile and/or a moderate manufacturing employment density.

3. Rural Area - The average population density is 1,000 persons per square mile. The rural area land use includes all population densities fewer than 2,000 persons per square mile.

Table B-III-C-1 shows the estimated and projected land use areas and their respective populations for 1970, 1980, 1990 and 2020.

VICINITY MAP

MICHIGAN
INDIANA

-N-

LAKE MICHIGAN

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⑰-I

⑲

PORTER CO
LA PORTE CO

LAKE CO
PORTER CO

ILLINOIS
INDIANA

LAKE CO

Figure B-III-C-I

C-SELM WASTEWATER
MANAGEMENT WATERSHEDS

5   0   5   10   15
SCALE IN MILES

B-III-C-2

DISTRIBUTION OF LAND USE AND POPULATION

Watershed Number	Description	Drainage Area (Sq.Miles)	1970 CONDITION						
			Land Use (Sq. Miles)			Population			
			Urban	Suburban	Rural	Urban	Suburban	Rural	Urban
1	Lake Michigan - North	60.6	2.0	21.0	37.6	20,000	84,000	18,000	2.0
2	North Branch Chicago River	90.1	2.0	35.0	53.1	20,000	140,000	27,500	2.0
3	DesPlaines River - North	260.9	2.5	41.0	217.4	25,000	164,000	102,750	2.5
4	Chicago Metro. Area	375.0	375.0	-	-	4,143,010	-	-	375.0
5	DesPlaines - Middle	80.0	16.0	32.0	32.0	160,000	128,000	17,000	16.0
6	Salt Creek	116.0	10.0	30.2	75.8	100,000	121,000	39,375	14.0
7	East Branch DuPage River	82.3	-	30.2	52.1	-	121,000	26,375	-
8	West Branch DuPage River	123.0	-	16.7	106.3	-	67,000	53,625	-
9	Main Stem DuPage River	194.0	-	-	194.0	-	-	51,750	-
10	San - Ship Canal - North	77.8	1.5	41.0	35.3	15,000	164,000	16,750	1.5
11	Cal - Sag Channel - North	52.4	2.5	29.3	20.6	25,000	117,000	9,625	2.5
12	San - Ship Canal - South	94.3	-	-	94.3	-	-	40,350	-
13	Cal - Sag Channel - South	40.7	-	10.5	30.2	-	42,000	16,250	-
14	Hickory - Spring Creeks	116.8	2.0	10.0	104.8	20,000	40,000	42,583	3.0
15	Jackson Creek	110.9	-	-	110.9	-	-	44,800	-
16	Thorn - Deer Creeks	110.3	12.0	37.0	61.3	120,000	148,000	31,000	13.0
17	Little Calumet - West[1]	34.1	-	-	34.1	-	-	6,000	-
17.1	Little Calumet - West[2]	32.6	-	7.5	25.1	-	30,000	12,250	-
18	Indiana Harbor	138.0	28.0	23.3	86.7	280,000	93,000	44,375	29.5
19	Little Calumet - Middle	141.6	-	5.0	136.6	-	20,000	55,010	-
20	Little Calumet - East	175.6	-	-	175.6	-	-	69,500	-
21	Indiana Dunes	46.1	-	-	46.1	-	-	50,600	-
22	Trail Creek	46.9	-	-	46.9	-	-	34,700	-
	TOTAL	2,600.0	453.5	369.7	1,776.8	4,928,010	1,479,000	810,168	460.5

Table B-III-C-1

OF LAND USE AND POPULATION

	970 CONDITION			1980 PROJECTION					
	Population			Land Use (Sq. Miles)			Population		
Rural	Urban	Suburban	Rural	Urban	Suburban	Rural	Urban	Suburban	Rural
37.6	20,000	84,000	18,000	2.0	31.0	27.6	20,000	124,000	13,000
53.1	20,000	140,000	27,500	2.0	49.5	38.6	20,000	198,000	20,250
217.4	25,000	164,000	102,750	2.5	57.8	200.6	25,000	231,000	94,375
-	4,143,010	-	-	375.0	-	-	4,174,700	-	-
32.0	160,000	128,000	17,000	16.0	45.5	18.5	160,000	182,000	10,250
75.8	100,000	121,000	39,375	14.0	50.3	57.7	140,000	201,000	27,375
52.1	-	121,000	26,375	-	47.5	34.8	-	190,000	17,750
106.3	-	67,000	53,625	-	33.7	89.3	-	135,000	45,125
194.0	-	-	51,750	-	-	194.0	-	-	78,100
35.3	15,000	164,000	16,750	1.5	55.3	21.0	15,000	221,000	9,625
20.6	25,000	117,000	9,625	2.5	37.0	12.9	25,000	148,000	5,750
94.3	-	-	40,350	-	4.0	90.3	-	16,000	48,000
30.2	-	42,000	16,250	-	16.5	24.2	-	66,000	13,250
104.8	20,000	40,000	42,583	3.0	23.0	90.8	30,000	92,000	40,000
110.9	-	-	44,800	-	-	110.9	-	-	51,900
61.3	120,000	148,000	31,000	13.0	49.0	48.3	130,000	196,000	24,500
34.1	-	-	6,000	-	-	34.1	-	-	6,800
25.1	-	30,000	12,250	-	9.2	23.4	-	37,000	11,375
86.7	280,000	93,000	44,375	29.5	25.2	83.8	290,000	101,000	42,875
136.6	-	20,000	55,010	-	8.0	133.6	-	32,000	71,900
175.6	-	-	69,500	-	-	175.6	-	-	98,300
46.1	-	-	50,600	-	-	46.1	-	-	58,500
46.9	-	-	34,700	-	-	46.9	-	-	29,200
776.8	4,928,010	1,479,000	810,168	460.5	542.5	1,597.0	5,029,700	2,170,000	818,200

DISTRIBUTION OF LAND USE AND POPULATION

Watershed Number	Description	Drainage Area (Sq. Miles)	1990 PROJECTION						Url
			Land Use (Sq. Miles)			Population			
			Urban	Suburban	Rural	Urban	Suburban	Rural	
1	Lake Michigan – North	60.6	3.0	37.0	20.6	30,000	148,000	9,500	5
2	North Branch Chicago River	90.1	2.0	63.0	25.1	20,000	252,000	13,500	3
3	DesPlaines River – North	260.0	2.5	88.0	170.4	25,000	352,000	79,500	12
4	Chicago Metropolitan Area	375.0	375.0	–	–	4,219,100	–	–	375
5	DesPlaines – Middle	80.0	16.0	55.0	9.0	160,000	220,000	5,500	17
6	Salt Creek	116.0	19.0	58.3	38.7	190,000	233,000	20,875	22
7	East Branch DuPage River	82.3	–	64.5	17.8	–	258,000	9,250	10
8	West Branch DuPage River	123.0	–	58.8	64.2	–	235,000	32,625	10
9	Main Stem DuPage River	194.0	–	9.2	184.8	–	37,000	85,875	8
10	San – Ship Canal – North	77.8	1.5	66.0	10.3	15,000	264,000	4,250	9
11	Cal – Sag Channel – North	52.4	2.5	43.0	6.9	25,000	172,000	2,750	5
12	San – Ship Canal – South	94.3	–	14.0	80.3	–	56,000	43,000	–
13	Cal – Sag Channel – South	40.7	–	24.5	16.2	–	98,000	9,250	–
14	Hickory – Spring Creeks	116.8	5.0	39.5	72.3	50,000	158,000	36,500	10
15	Jackson Creek	110.9	–	4.5	106.4	–	18,000	51,750	2
16	Thorn – Deer Creeks	110.3	14.0	54.5	41.8	140,000	218,000	21,200	20
17	Little Calumet – West[1]	34.1	–	–	34.1	–	–	15,500	–
17.1	Little Calumet – West[2]	32.6	1.0	9.0	22.6	10,000	36,000	11,000	2
18	Indiana Harbor	138.0	32.0	53.5	52.5	320,000	214,000	27,250	33
19	Little Calumet – Middle	141.6	3.0	10.5	128.1	30,000	42,000	64,750	5
20	Little Calumet – East	175.6	–	15.0	160.6	–	60,000	79,000	10
21	Indiana Dunes	46.1	–	12.5	33.6	–	50,000	16,750	2
22	Trail Creek	46.9	–	3.5	43.4	–	14,000	22,100	–
	TOTAL	2,600.0	476.5	783.8	1,339.7	5,234,100	3,135,000	661,675	565

1990 PROJECTION				2020 PROJECTION					
	Population			Land Use (Sq. Miles)			Population		
Rural	Urban	Suburban	Rural	Urban	Suburban	Rural	Urban	Suburban	Rural
20.6	30,000	148,000	9,500	5.0	43.5	12.1	50,000	174,000	5,250
25.1	20,000	252,000	13,500	3.0	68.5	18.6	30,000	274,000	10,250
170.4	25,000	352,000	79,500	12.5	122.0	126.4	125,000	488,000	57,250
-	4,219,100	-	-	375.0	-	-	4,341,100	-	-
9.0	160,000	220,000	5,500	17.0	63.0	-	170,000	260,000	-
38.7	130,000	233,000	20,875	22.0	70.5	23.5	220,000	282,000	13,250
17.8	-	258,000	9,250	10.0	65.0	7.3	100,000	260,000	4,000
64.2	-	235,000	32,625	10.0	90.5	22.5	100,000	362,000	11,750
184.8	-	37,000	85,875	8.0	35.5	150.5	80,000	142,000	68,750
10.3	15,000	264,000	4,250	9.5	66.5	1.8	95,000	266,000	-
6.9	25,000	172,000	2,750	5.5	42.5	4.4	55,000	170,000	1,500
80.3	-	56,000	43,000	-	34.2	60.1	-	137,000	32,875
16.2	-	98,000	9,250	-	32.2	8.5	-	129,000	5,375
72.3	50,000	158,000	36,500	10.0	69.2	37.6	100,000	277,000	18,875
106.4	-	18,000	51,750	2.0	15.7	93.2	20,000	63,000	45,125
41.8	140,000	218,000	21,200	24.0	83.0	3.3	240,000	332,000	2,000
34.1	-	-	15,500	-	0.8	33.3	-	-	16,000
22.6	10,000	36,000	11,000	2.0	14.0	16.6	20,000	56,000	8,000
52.5	320,000	214,000	27,250	33.0	56.0	49.0	330,000	224,000	25,500
128.1	30,000	42,000	64,750	5.0	24.5	112.1	50,000	98,000	56,750
160.6	-	60,000	79,000	10.0	49.0	116.6	100,000	196,000	57,000
33.6	-	50,000	16,750	2.0	18.7	25.4	20,000	75,000	12,625
43.4	-	14,000	22,100	-	9.7	37.2	-	39,000	19,625
1,339.7	5,234,100	3,135,000	661,675	565.5	1074.5	960.0	6,246,100	4,304,000	471,750

STORMWATER RUNOFF PROJECTION METHODOLOGY

Stormwater runoff from urban areas is estimated by utilizing 21 years of stormwater data (1949-1969) for the 375 square mile combined-sewered area of the City of Chicago. $\underline{1}/$ Analysis of this data indicates an average yearly runoff from urban areas equal to 19 inches.

Stormwater runoff from rural areas is estimated through the use of the United States Geological Survey (U.S.G.S.) streamflow data for five predominantly rural watersheds within the C-SELM area. The selected watersheds are Hickory Creek, Long Run, DuPage River, Deep River and Burns Ditch. $\underline{2}/$ $\underline{3}/$ The period of record for these five watersheds ranges from 19 to 30 years. Analysis of this historical streamflow data indicates an average yearly runoff from rural areas equal to 10 inches.

For the suburban areas, it is estimated that the average yearly stormwater runoff approximates 12 inches.

The total stormwater runoff is then projected through the use of the above-mentioned factors and the respective land development classifications. Presented in Table B-III-C-2 are the pertinent average daily *stormwater* flows for both present and future design years (1990 and 2020) based on the land use classification. The total flows are further broken down into that portion of stormwater which infiltrates into separate and combined sewered systems and that portion which under present day circumstances runs off directly to streams in the C-SELM area.

Table B-III-C-2

ESTIMATED STORMWATER RUNOFF FLOWS
(Average Daily Flow in MGD)

| | Design Year | | |
Stormwater Component	1970	1990	2020
Total Runoff	1,456	1,529	1,582
- Urban	405	436	510
- Suburban	205	455	615
- Rural	846	638	457
Urban-Suburban Infiltration			
to Municipal Sewer Systems	300	365	422
Urban-Suburban Runoff			
Directly to Streams	310	526	703

As can be seen from the table, stormwater runoff increases in direct proportion to areas being developed from rural to suburban and suburban to urban. Thus, as more area is converted from natural soil and vegetation coverage to impervious, built-up land, the annual stormwater runoff is projected to increase.

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III.   FLOW BASIS OF DESIGN

D.   SUMMARY OF PROJECTED TOTAL FLOWS

        A summary of the present and projected 1990 and 2020 waste-
water and stormwater flows for the C-SELM region is presented in
Table B-III-D-1.   These flows are classified into two major parts:
(1) sewage treatment plant flows which are comprised of domestic-
commercial wastewater, together with infiltrated stormwater, and those
industrial flows which discharge to municipal systems and (2) the
direct discharges, which include those industrial flows that discharge
effluent directly to watercourses in the C-SELM area, and stormwaters
which runoff directly to the C-SELM streams located in urban, subur-
ban or rural areas.   The projected flows to the municipal treatment
systems include all flows except runoff from rural areas which is treated
through the rural stormwater management system.   These municipal treat-
ment plant flows are projected to total 3,000 MGD in 1990 and 3,630
MGD in 2020.

Table B-III-D-1

PRESENT AND PROJECTED C-SELM
WASTEWATER AND STORMWATER FLOWS
(Average Daily Flow in MGD)

Flow Component	Design Year		
	1970	1990	2020
Sewage Treatment Plants			
Domestic-Commercial	700	870	1,300
Industrial	600	695	765
Stormwater Infiltration	300	365	422
Sub-Total	1,600	1,930	2,487
Direct Discharges			
Industrial	2,850	544	440
Urban-Suburban Stormwater			
Runoff	310	526	703
Rural Stormwater Runoff	846	638	457
Sub-Total	4,006	1,708	1,600
TOTAL C-SELM FLOWS	5,606	3,638	4,087
Projected Municipal			
Treatment System Flows	-	3,000	3,630
		(3638-638)	(4087-457)

TECHNICAL APPENDIX B

COMPONENT BASIS OF DESIGN

IV.   COMPONENT BASIS OF DESIGN

A.   REGIONAL TREATMENT SYSTEMS

INTRODUCTION

Detailed design information regarding the key treatment facility components is presented in this section. These treatment components are structured so as to comprise the three advanced wastewater treatment (AWT) systems which are utilized in this study for the achievement of the ultimate water quality goals.

These regional AWT treatment systems are designed to treat domestic, commercial and industrial wastewaters together with regulated stormwater flows for the achievement of the NDCP effluent goals. The character of the wastewater, whose definition is necessary for the detailed design of the AWT components, is a function of the type and amount of the particular waste. For example, stormwater is generally less concentrated in terms of phosphorus and nitrogen loadings than domestic wastes; however, its suspended solids loading can be far greater. The estimated overall influent wastewater characteristics for the C-SELM service area are presented in Table B-IV-A-1 for a number of key wastewater parameters.

TREATMENT PLANT SYSTEMS

Treatment Capacity and Storage

Because of the expensive hardware for the advanced treatment plant systems, it was determined through a cost analysis that the optimum AWT design would be accomplished by means of flow regulation so as to minimize peak flows through the plants. Peak treatment plant flows are mainly caused by two factors. The first and most significant factor, in terms of cost and performance of AWT systems, is peak flow contributed by stormwaters entering the municipal system. As is discussed in detail in Appendix B, Section IV-D, the peaking effect of stormwater runoff is controlled through the utilization of on-site storage of stormwater prior to treatment at the plant. The on-site storage regulates peak storm flows and enables an optimum pump-out rate of stormwaters to AWT facilities for treatment. The second peaking factor is related to domestic water consumption trends which result in a vary-

Table B-IV-A-1

PROJECTED RAW WASTEWATER CHARACTERISTICS
FOR THE C-SELM AREA

Parameter	Influent Concentration (mg/l)
Total Dissolved Solids	600
Suspended Solids	200
Chemical Oxygen Demand	350
Biochemical Oxygen Demand (5-day)	150
Total Nitrogen as N	30
Total Phosphorus as P	10

ing sanitary sewage flow throughout any one day. Low flow typically occurs during early morning hours while peak diurnal flow occurs during daylight hours. The ratio of the peak to average daily flow is a function of the population served by a facility. As the service population increases the ratio of peak to average flow decreases. This concept is graphically presented in Figure B-IV-A-1 by the Harmon equation. 1/

The plant capacity for the treatment plant systems is equal to the sum of the average daily dry weather flow plus regulated flow from stormwater storage facilities. If the peak diurnal flow exceeds the above-mentioned plant capacity, then storage is provided at the plant site to regulate these peaks. The storage provided is equal to four times the difference between the peak diurnal flow and the plant capacity (average dry weather flow plus stormwater pump-out). This volume of storage is provided since the sewer system, which is sized for peak diurnal flows, will flow full during wet weather conditions (stormwater infiltration) for periods of up to four days after the storm event. One exception to this methodology is made with respect to the three large Metropolitan Sanitary District of Greater Chicago (MSD) plants which serve the combined-sewered area of the City of Chicago. For these plants, the capacity is designed for 1.5 times the average dry weather flow in accordance with present stormwater plans for the City.

The factors used to determine the component or unit process design capacity recognize the influence of stormwater and infiltration that enter the system as well as the recirculated flows resulting from advanced process units, such as backwash from filters and incinerator wet scrubbers.

In determining these unit process design flows, a recirculation factor and a peaking applicability factor are developed. The recirculation factor indicates the portion of flow that is required to be recirculated or recycled through the process unit. The peaking applicability factor designates that portion of the process unit which is sensitive to variations in flow above the average and for which peak flow design is necessary.

For treatment plant systems, the unit process design flow is determined by the following equation:
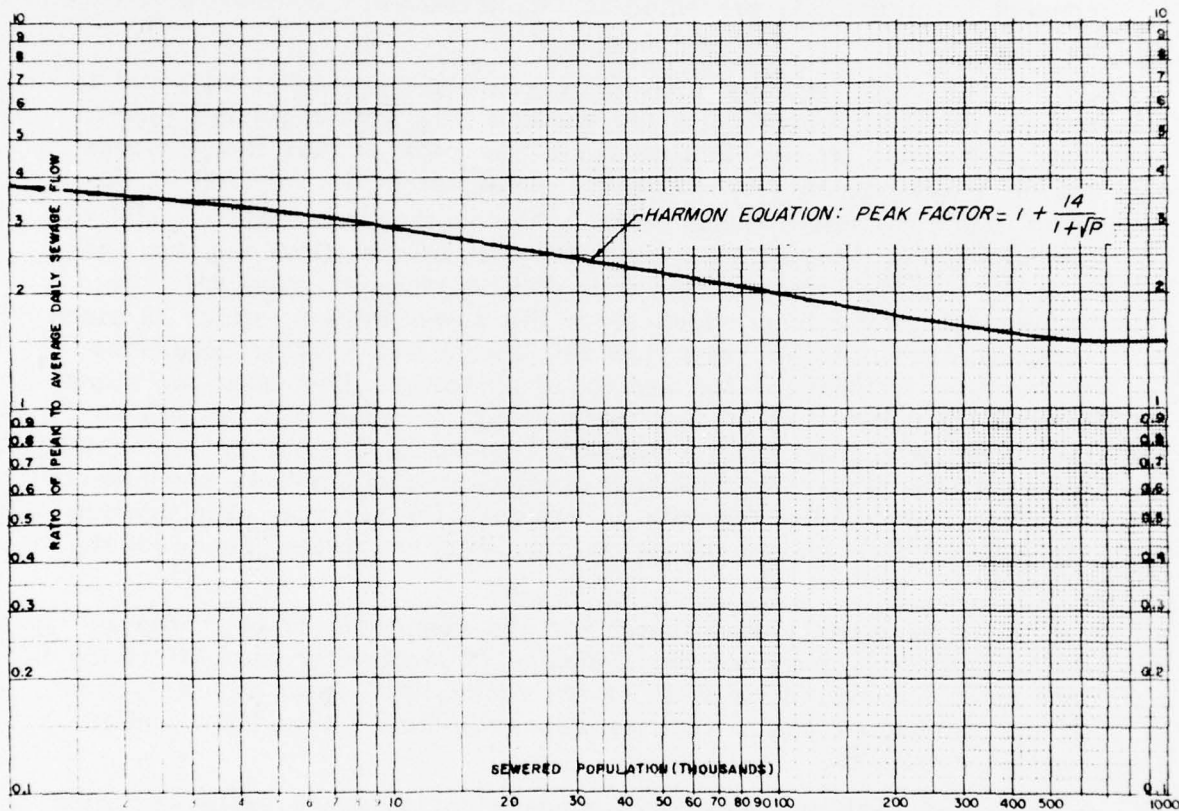
Figure B-IV-A-1

PEAK FLOW FACTORS FOR
DOMESTIC WASTEWATERS

Unit Process Design Flow = $DWF(RF) \left[1 + (PF-1)(PAF)\right]$

where DWF = dry weather flow
  RF = recirculation factor
  PF = peaking factor = peak flow ÷ dry weather flow
  PAF = peaking applicability factor

When stormwater on-site storage is provided, the peaking factor for the AWT plants (not including the three MSD facilities)　is equal to the plant capacity (average dry weather flow plus storm pump-out) divided by the average dry weather flow.

  As is presented in the cost evaluation section of Appendix D, the AWT systems are analyzed under two conditions.　In the first analysis, only that portion of stormwater which infiltrates into the sewer system servicing a plant is treated.　Although the O & M costs are based on flows without stormwater, the treatment facilities are designed to accommodate the eventual phase-in of complete stormwater treatment.　For the without-stormwater analysis, the PAF is less than one for certain unit processes.　This takes into account the fact that certain treatment unit capacities are not affected by peak flows over short periods of time such as would occur in this analysis from stormwater infiltration during wet weather conditions.　In the second analysis, all stormwater runoff generated in the service area is treated. Peak flows in this analysis could occur over significant time periods since continuous stormwater pump-out from storage facilities would be necessary for one or two month periods.　Since the peak flows occur for such lengthy time periods, the PAF of all unit processes must equal unity to insure proper performance.　Presented in Table B-IV-A-2 are the PAF and RF values for the various treatment plant system process units.

  An example calculation for determining the design capacity of an ion exchange unit follows:

  DWF = 10 MGD
  Stormwater Pump-out Rate = 10 MGD
  RF = 1.1
  PAF = (without stormwater analysis) = 0.75
  PAF = (with stormwater analysis) = 1.0
  PF = $\dfrac{10 + 10}{10}$ = 2.0

Table B-IV-A-2

TREATMENT PLANT DESIGN FLOW DATA

Treatment Process	Peaking[a] Applicability Factor	Recirculation Factor
Ammonia Removal by Ion Exchange	0.75	1.10
Carbon Absorption and Regeneration	0.75	1.30
Chlorination	1.00	1.00
Multimedia Filtration	1.00	1.10
Nitrification - Denitrification	1.00	1.00
Phosphorus Removal - 98%	1.00	1.40
Post Aeration (1 mg/l to 6 mg/l D.O.)	1.00	1.00
Primary and Biological Secondary (including Sludge Stabilization and Storage, Chlorination)	0.75	1.00

[a] For onsite storage of stormwater, and hence regulated stormwater to treatment facilities, the PAF = 1.0 for all process units.

For the without-stormwater analysis, the unit process design flow is calculated as follows:

Unit Design Flow = 10 (1.1) $\left[1 + (2-1)(.75)\right]$ = 19.25 MGD

For the with-stormwater analysis, the unit process design flow is calculated as follows:

Unit Design Flow = 10 (1.1) $\left[1 + (2-1)(1)\right]$ = 22 MGD

In this example, although the plant capacity is equal to 20 MGD (average dry weather flow plus stormwater pump-out), the ion exchange unit in the without-stormwater analysis has a capacity of 19.25 MGD while the same unit in the with-stormwater analysis has its capacity increased to 22 MGD.

Treatment Plant Component Design Parameters

Introduction. The treatment plant systems are designed in detail based on a modular 100 MGD plant capacity. This plant size is studied since it represents the point at which no further economies of scale for treatment facilities are projected. It should be noted that the individual capacities of the treatment plant components may exceed the capacity of the system due to the flow recirculation. Thus certain unit processes have capacities in the range of 110 MGD to 140 MGD for the modular design.

Advanced biological treatment utilizes presently available technology to achieve an NDCP performance level. This technology integrates existing conventional secondary treatment plants which provide primary and secondary removal of solids and organics from the waste-waters with tertiary AWT components. Nitrogen control is accomplished biologically by a two-sludge system.

Physical-chemical treatment utilizes unit processes which have generally been used by various industries for specific wastewater treatment applications. These processes have been adapted into an AWT design to again attain the NDCP goal. Unlike the advanced biological technology, there can be little adaptation of existing secondary facilities into this system. Hence, essentially complete replacement of existing facilities is necessary. This technology differs from advanced biological in that the majority of solids and organics are removed by the lime clarification process, while nitrogen control is accomplished through the use of an ion exchange process.

Conventional secondary treatment. This system utilizes biological and physical processes to accomplish the secondary treatment of wastewater. The processes include screening of the influent sewage prior to arrival at the raw sewage lift station. Wastewater is then pumped to aerated grit tanks for grit removal and then to primary settling basins for heavy solids removal Primary tank effluent then flows to the aeration tanks where biologial stabilization of the wastewater takes place. There the soluble organic concentration in the wastewater is markedly reduced by biological synthesis and respiration. The mixed liquor passes from the biological reactor to the secondary clarifier where the majority of the settleable solids are removed. Effluent from the final settling tank then flows to the chlorine contact tank where chlorination of the secondary treated water is accomplished prior to discharge to receiving streams. The majority of the settled activated sludge from the final settling tank is returned to the aeration tank to insure an active biological population for treatment of the wastes. The remainder of the settled sludge from the final tank is wasted and combined with the solids underflow from the primary tank for high-rate anaerobic sludge digestion. The digested solids are then pumped to sludge dewatering lagoons prior to final sludge utilization.

Mechanical bar screens are designed for collecting some 100 cubic feet of screenings per day for the modular 100-MGD facility. Three screens, each with a capacity of 35 MGD over a channel width of 5 feet, are employed. The screens are designed with 3/4" openings, and a belt conveyor drive is utilized for disposal of screenings by truck to landfills. The sewage lift station is designed to accommodate the plant capacity flows (equal to average dry weather flow plus stormwater pump-out). It is assumed that a typical lift station would have five 17,500-gpm mixed-flow pumps, with four in operation and one standby unit together with adequate emergency power generation facilities. The total dynamic head (TDH) requirements for a raw sewage lift station are in the range of 30 to 50 feet.

Aerated grit tanks are also designed on a modular basis for a 100 MGD flow. Wastewater retention time is 4 minutes with air supplied through diffuser tubes at 6 cfm per foot of tank length. A typical grit tank dimension is 20 feet wide, 40 feet long and 12 feet deep, with weir controls. For the 100 MGD plant, four such units are necessary for adequate grit removal. The grit removal facilities consist of grit screw conveyors to bucket elevators to a belt conveyor which dis-

charges the grit to two grit storage hoppers. These hoppers are designed for each to accommodate the grit generation of one day. The grit is then discharged to trucks for disposal to landfill or to sludge lagoons for mixture with the digested solids.

Solids loading to the primary settling tanks is estimated at 83 tons/day (200 mg/l suspended solids @100 MGD). The tanks are designed to remove 60% of the suspended solids during a 2-hour wastewater detention time. These tanks are rectangular units with dimensions approximating 40 feet in width, 145 feet in length and 10 feet in depth. For the 100 MGD modular design, 20 primary units are necessary. The design overflow rate is 900 gpd/square foot. The primary sludge is designed to be withdrawn at a 3-4% solids concentration. Thus, for each primary unit a raw sludge collector drive and pump with a capacity of 250 gpm is required. These pumps are designed for a typical TDH of some 80 feet dependent on the particular plant layout. Settling tanks also utilize ejectors with air compressors with a capacity of 150 gpm and a TDH of some 50 feet. Finally, the weir loading for the primary units is designed at 15,000 gpd/foot.

Aeration tank design utilizes the plug-flow treatment concept with aeration being provided through diffuser tubes. The average modular design flow is 100 MGD with a detention time of six hours. Aeration tank unit dimensions are 100 feet wide, 200 feet long and 14 feet deep. Plug-flow design is accomplished by 4 passes per tank. For the 100 MGD design flow, 12 aeration tanks are necessary with overflow rates equal to 420 gpd/square foot. Based on a 150 mg/l BOD_5 influent loading and a 30% BOD_5 removal in the primary treatment unit, the BOD_5 loading to the aeration tank is approximately 25 pounds per 1,000 cubic feet of tank capacity. The return activated sludge flow, which is equal to 50% of the average design flow, increases the loading to 35 pounds per 1,000 cubic feet of capacity. Based on a blower design requirement of 1,500 cubic feet of air per pound of BOD_5 removed, the air requirement is approximately 80,000 standard cubic feet per minute (scfm). Therefore, three 40,000 scfm blowers are designed for the system with one standby and two operating units.

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The final clarifiers, or settling tanks, are designed for an over-flow rate of 800 gpd/square foot. For 100 MGD design flow, twelve tanks 10 feet deep and 115 feet in diameter are necessary. The waste-water detention time for these tanks is 2.3 hours. As mentioned previously, the return activated sludge flow is designed for 50% of the average flow or for this modular design, 50 MGD. Three (two operating, one standby) return-sludge, variable-speed pumps, each with capacity of 25 MGD, are utilized for this design. The TDH for these pumps is estimated at 40 feet. Each clarifier unit has a sludge collector with a scum-skimming drive. Assuming a total suspended solids removal in the range of 85-90% for secondary treatment and a 60% solids removal in the primary unit, the estimated waste solids removal from the secondary clarifier is approximately 23 tons/day for the 100 MGD design flow. Assuming a waste activated solids content in the range of 0.5 to 1.0%, the waste activated sludge rate is some 450 gpm. Two pumps (one variable speed and one constant speed) each with a rated capacity of 450 gpm and a TDH of 80 feet are designed for the waste sludge pumping.

Primary and waste activated sludges are pumped to anaerobic digesters utilizing a high rate design. This design includes continuous solids feed and displacement, complete mix of digester solids and heating of the tank to optimal digestion temperatures (mesophilic temperatures in the range of 80-100°F). The detention of the digested sludge is 15 days, which relates to a tank capacity of approximately 1.5 cubic feet per capita. Digestion is designed to provide a volatile solids reduction of 60%. Approximately 75% of the total solids are volatile. Sludge yield from the digesters is estimated at 0.73 tons per MG of wastewater flow, through the plant. For the 100 MGD modular design, five high-rate digester units are designed with four in operation and one standby unit. These digesters are 150 feet in diameter and 20 feet deep with fixed covers and gas collection equipment including gas storage, distribution and waste-gas burner devices. Assuming a sludge digestion design temperature of 95°F and rete digester construction, the estimated heat requirements, including heat losses from the digester, total some 3.72 million BTU/day/MGD of average wastewater treatment plant flow. Utilizing typical digester design criteria of: digester gas yield equal to one cubic foot per capita per day; methane gas content equal to 72%; and a methane heating value equal to 960 BTU per cubic foot, it is estimated that all digester heating requirements can be completely met by use of the digester gas itself.[2] Digester heat distribution is accomplished by means of hot water circulating through fixed or moving coils in the tank.

The digested solids are then pumped to sludge detention lagoons for the purpose of dewatering and winter storage (four months) prior to conveyance to sludge utilization areas.

Nitrification-denitrification. This unit process consists of a two-stage biological reactor system for the control of soluble nitrogen in wastewater. This system is a part of the advanced biological process. In the nitrification reactor, autotrophic nitrifying bacteria (Nitrosomonas and Nitrobacter groups) oxidize ammonia nitrogen to the nitrite and finally to the nitrate form. Since the nitrification process (oxidation or ammonia) is essentially a linear or zero-order reaction, short-circuiting within the treatment tank must be eliminated. Therefore, the nitrification tank configuration is designed for plug-flow mixing.$\underline{3}/$ The control of pH is an important design consideration for the nitrification process. In the nitrification process, alkalinity if destroyed results in lowered pH levels which greatly inhibit the nitrifying bacteria. The nitrification system is designed for a pH of 7.5 (optimum nitrification rates occur in the pH range of 8.5). Adequate mixing and aeration is provided in this process to drive off carbon dioxide to the atmosphere thereby increasing the pH. If the alkalinity of the wastewater is not sufficient, provisions should be made for lime additions from the lime recalcination process. Detention time for the nitrification unit is three hours. For the 100 MGD modular design, six nitrification tanks are necessary. The dimensions of these units are 100 feet wide, 200 feet long, and 14 feet deep with four passes in each tank for plug-flow. Aeration requirements are based on 4.6 pounds of oxygen per pound of influent ammonia plus 1 pound of oxygen per pound of influent BOD. For a typical secondary effluent with a BOD_5 equal to 20 mg/l, a NH_3-N concentration equal to 17 mg/l, and blower requirements of 1,500 cubic feet of air per pound of oxygen demand, theoretical aeration requirements are approximately 85,000 scfm for the 100-MGD design. Three blowers are used (two operation and one standby) for this diffused air system, each with a capacity of some 40,000 scfm.

Nitrification clarifiers are designed for a 3.2-hour wastewater detention time (without return sludge flows). The average hydraulic surface loading is approximately 650 gpd/square foot for average daily flows and 1,000 gpd/square foot for return flows which are 50% of the average flow. In order to accommodate the 100-MGD modular design flow, 12 tanks 12 feet deep and 126 feet in diameter are necessary. Three 25-MGD return sludge pumps (two operation and one standby)

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with A TDH of some 40 feet are used in this design. Clarifiers are also equipped with skimmers and scum ejectors to pump floating sludge, should it ever occur, to the head of the nitrification unit. Because of low cell synthesis in the nitrification process, it is projected that little sludge will be waste.[4] However, for flexibility in system control concerning mixed-liquor suspended solids (MLSS) concentration, a 75 gpm waste activated sludge pump is utlized in the system design for wasting the sludge to the secondary digester.

Anaerobic conditions are required for the denitrification stage in which common facultative heterogrophic bacteria reduce the nitrates to a nitrogen gas end product. The design of this process is also based on the plug-flow mixing model due to short detention times and the inability of nitrates to be absorbed by biological growths. Wastewater detention time for this unit is one hour. Dimensions for the denitrification tank are 100 feet wide, 200 feet long, and 14 feet deep with four passes per unit for plug-flow. Two such tanks are necessary for handling the modular design flow of 100 MGD. Denitrification tanks are to be covered to minimize oxygen absorption from the atmosphere to maintain anaerobic conditions. A methanol feed system is also designed to provide a readily available carbon source for active denitrifying organisms. The methanol feed requirement is based on the commonly used equation: Methanol feed in mg/l = 2.47 NO_3-N mg/l + 1.53 NO_2-N mg/l + 0.87 O_2 mg/l. Based on an influent NO_3-N concentration of 15 mg/l, a dissolved oxygen content of 2 mg/l and essentially no NO_2-N, the design methanol feed is established at 40 mg/l or some 5,100 gallons of methanol per day for the 100-MGD design. Methanol storage tanks are designed to handle a two-week supply of design feed.

The basis of design for denitrification settling tanks is the same as that previously mentioned for nitrification clarifiers. Return sludge is equal to 50% of the average design flow. Waste sludge is generated in the denitrification clarifiers. It is reported that approximately 0.2 pounds of sludge is generated for every pound of methanol feed. [3] For a 100-MGD design flow, this results in a dry solids generation of 6,700 pounds per day which can be wasted to the secondary digester by means of a 75-gpm pump.

Lime clarification. The lime clarification process is utilized by both the physical-chemical and the advanced biological treatment systems. The only difference in the design of this unit process is in the solids handling section. For the physical-chemical system, there are approximately twice as many solids, by dry weight, entering the incinerator than in the advanced biological system. In the lime clarification process, the lime is slaked and fed into a flash mixer with influent wastewater. The high pH wastewater is discharged to reactor clarifiers where flocculation and sedimentation occur. Settled solids are then pumped to sludge thickeners and vacuum filters in order to increase the solids concentration. This lime sludge is then discharged to a fluidized-bed incinerator for the purpose of lime recalcination. Furnace ash is air classified for separation of impurities from the recalcined lime. The recalcined lime is then reslaked and pumped to the lime clarification influent wastewater. The remaining solids are discharged to trucks for transport to sludge utilization areas or mixed with water for pipeline transmission to these areas. Carbon dioxide from the lime furnace stack gases is collected and compressed. The CO_2 is then diffused into two-stage recarbonation units for pH control. By decreasing the pH in the first stage, calcium hydroxides are converted to calcium carbonate which results in a dense, rapidly-settling sludge. This sludge is then added to the lime clarifier sludge for lime recalcination. In the second stage recarbonation, further CO_2 is diffused into the wastewater to accomplish the design pH of 7.

From an advanced treatment standpoint, the main purpose of the lime clarification process is the removal of phosphorus. In order to meet the NDCP phosphorus goal of less than 0.2 mg/l, this necessitates 98% phosphorus removal for a typical wastewater influent phosphorus concentration of 10 mg/l. To accomplish this high degree of phosphorus removal, a high pH in the wastewater is necessary to precipitate out the soluble phosphorus. [5] In order to accomplish a pH of 11, a lime dosage of 400 mg/l is programmed.

This dosage is applied to the total flow (including recirculation flows) through the lime process, which for our modular 100-MGD plant is 140 MGD. This amounts to a daily utilization of some 233 tons of lime for the modular design. Of this total, 72% is recalcined and recycled; the remaining 28%, or 64 tons of lime, must be made up every day. Provisions are made for 15-day storage of lime make-up needs which equals some 1,000 tons of lime. This lime is stored in ten storage bins 40 feet high and 12 feet in diameter. The lime is conveyed from the storage facilities to slakers by means of a 12-inch screw conveyor with a rated capacity of 15-20 tons per hour. A slaker, 8-feet deep and 11 feet in diameter, with a rated capacity of 235 tons/day, is utilized in this design. An identical slaker is also provided for back-up purposes. Typical slaker detention time is 20 minutes and the water/lime ratio is 3.5 to 1. $\underline{6/}$ Slaked lime then proceeds to flash-mixing basins where it is added to influent wastewater. Designing for a velocity gradient, G, equal to 700 fps/foot and a detention time of 30 seconds at standard temperatures, the required horsepower for mixing equals 1 HP/MGD. $\underline{7/}$ For the 140 MGD modular design flow, two rapid-mix basins are designed (one backup) each with a capacity of 140 MGD and equipped with 140-HP mechanical mixers. The dimensions of these basins are approximately 20 feet square and 14 feet deep.

The lime clarification step is accomplished in four reactor clarifiers 17 feet deep and 220 feet in diameter. At the 100-MGD modular flow, the overflow rate is some 670 gpd/square foot; this increases to over 900 gpd/square foot for the additional 40 MGD of recirculated flows. Approximately 70% of the physical-chemical solids are collected in the lime clarification reactor. This amounts to over 700,000 pounds of dry solids/day for the modular design. The solids content of this sludge is approximately 10%; therefore, sludge pumps with a total rated capacity of some 600 gpm are necessary for the physical-chemical design. For the advanced biological system, the majority of suspended solids are removed and treated by secondary facilities. For this system, lime clarification solids loading is decreased to some 550,000 pounds of dry solids/day (470 gpm @10% solids content). This is equivalent to 65% of the solids collected in the lime clarification process of the advanced biological system. From the lime clarification process, high pH wastewater proceeds to a two-stage recarbonation process. In this step, four recarbonation clarifiers 17 feet deep and 220 feet in diameter are used for the modular design. The overflow rate is the same as for the lime clarifiers. In the first stage, carbon dioxide is diffused into

the wastewater to decrease the pH to 9.3 which corresponds to the minimum solubility of calcium carbonate. Sludge is then pumped to the solids-handling system of the lime recalcination process. In the second stage, further carbon dioxide is added to decrease the pH to 7.0. Carbon dioxide requirements are based on the alkalinity of the wastewater. For wastewaters in the high pH range of 11, it is estimated that, at standard conditions (14.7 psia and 60°F), the compressor capacity necessary will be some 190 cfm/MGD of wastewater treated. This assumes a CO_2 stack gas content of 10% at a density of 116 pounds per 1,000 cubic foot, a CO_2 requirement of 2,800 pounds per MG and a temperature of 110°F from the scrubbing waters.[5/8/] Approximately 27,000 cfm of compressor capacity is necessary for this modular design. This is accomplished through the use of four 9,000-cfm capacity compressors with three in operation and one standby unit. The calcium carbonate sludge generated in the first-stage reactor-clarifier exceeds approximately 300,000 pounds of dry solids/day for the modular design. This quantity is the same for both treatment plant systems and the solids content is assumed at 15%. In order to handle this sludge generation, sludge pumps with a total capacity of 200 gpm are necessary with duplicate backup pumps designed into the system because these lime sludges form quite readily and extensive down-time must be prevented. Approximately 75% of the CO_2 is utilized in the first stage and the remainder is diffused into the second-stage recarbonation units.

Sludge from the recarbonation basins is combined with the sludge generated from the lime clarification units and proceeds to sludge thickeners. For the physical-chemical systems, the dry solids fed to the thickeners approximates 1.1 million pounds/day for the modular treatment facility design. The design solids loading rate is 50 pounds/day/square foot. This requires the use of six thickeners 12 feet deep and 70 feet in diameter. The projected sludge solids content from the thickeners is 23%; this sludge is withdrawn at a rate of 400 gpm. For the advanced biological system, approximately 0.85 million pounds of dry solids/day are fed into the thickeners. At the design loading rate of 50 pounds/day/square foot, six thickeners 12 feet deep and 60 feet in diameter are required. Withdrawal pumps discharge the 23%-solids-content sludge at a rate of some 300 gpm.

The thickened sludge is then sent to vacuum filters for dewatering prior to recalcination. The filter loading rate is designed at 8 pounds of dry solids/hour/square foot. For the physical-chemical system, which produces 46,000 pounds of solids/hour, eight filters-12 feet

in diameter are required, each with a filter face of 20 feet. Based
on a vacuum pump air-flow-rate design of 3 cfm/square foot, each
vacuum filter requires a 2,250 cfm vacuum pump. Filtrate is dis-
charged to the lime recalcination unit at a rate of 12,200 gph with
a design solids content of 45%. For the advanced biological system,
total dry solids generation is decreased to 35,000 pounds/hour. Based
on the same design-solids loading-rate of 8 pounds/hour/square foot,
six vacuum filters, which are identical to those described above for
the physical-chemical system, are required.

The lime sludge cake is then discharged from the filters via
a screw conveyor to a fluidized-bed incinerator for lime recalcination.
At temperatures of 1600°F, the solids are burned to ash. The off
gases are water scrubbed prior to release to the atmosphere. The
inert solids are removed from the scrubber flow by a cyclone-separa-
tor. The solids are then discharged by a screw feeder to a lump
breaker and thermal disc cooler. The solids are then rotary fed to
an air classifier which separates out the majority of inert solids from
the recalcined lime. 9/ The air-classified lime is then sent to a
cyclone for solids separation prior to being sent to lime storage faci-
lities.

The lime feed to the fluidized-bed incinerators in the advanced
biological system is 233 dry tons/day for the modular design. Based
on a two-hour inert solids retention time in the feed for complete re-
calcination and a dry-solids feed of 36,000 pounds/hour, two 11-foot
diameter fluidized-bed incinerators are required. Based on the same
two-hour inert-solids retention time and a physical-chemical dry-solids
feed of 46,000 pounds/hour, two-14 feet diameter incinerators are re-
quired for the physical-chemical system. For both systems, an addi-
tional incinerator is utilized for backup purposes. The static bed
height of these incinerators is seven feet and they are designed to ex-
pand to a ten-foot height. Freeboard bed diameters for these units
range from 18 to 23 feet. Fluidizing capacities of 4,000 to 6,000 scfm
are provided by 250 horsepower blowers.

Dry-solids feed to the air classifier from the physical-chemical
system is approximately 0.6 million pounds/day for the modular design.
In order to accommodate this load, two 15,000 pounds/hour capacity
air classifiers are utilized. The inert rejects from this classifier
amount to some 1.1 tons of dry solids/MG of wastewater treated. The
lime recalcination system is designed to recover approximately 70%
of the lime dosage. For the advanced biological system, 0.5 million
pounds of dry solids are fed into the air classifiers. This

requires two 12,000 pound/hour capacity air classifiers. The inert rejects from this system approximate 0.85 tons of dry solids/MG of treated wastewater. The total recirculated flow from the lime clarification process is estimated at 10 MGD for the modular design. The majority of this flow is contributed by the wet scrubbers from the lime recalcination unit. Minor amounts are also contributed by the thickening and sludge-dewatering operations. All recirculated flows are returned to the influent lime-clarification wastewater. Recirculated flows from the carbon adsorption process and the mixed-media filtration process are also sent to the head of the lime clarification process, thereby making the total recirculated flow through the process equal to 40 MGD.

Carbon adsorption. The carbon adsorption system designed for this study utilizes granular carbon and has application in both the physical-chemical and the advanced biological systems. Granular activated carbon has the characteristic of adsorbing dissolved organic material rather completely from wastewater. For both technologies, the carbon adsorption system follows the lime clarification step. A major control on the performance of this unit process is the wastewater detention or carbon contact time. Although the organic loading to the columns is much greater in the physical-chemical system, the contact time and design loading rate require the same number of carbon contactors for both technologies. The difference between the two systems is that the carbon dosage required to treat the physical-chemical wastewater is twice that of the advanced biological system, thereby necessitating larger carbon regeneration facilities for the physical-chemical system.

In the physical-chemical system, the estimated COD effluent concentration for the lime clarification process is 70 mg/l. The carbon adsorption isotherm is such that as the influent organic loading is increased, the adsorptive capacity of the carbon increases. The estimated adsorptive capacity of the carbon is 0.75 pounds of COD/ pound of carbon.[10] This adsorptive capacity can be influenced greatly by biological growths on the carbon particles. Using this adsorptive capacity and an overall COD removal of 60 mg/l from this process results in a design capacity for the physical-chemical carbon adsorption system of 700 pounds of carbon-MG of treated wastewater. For the advanced biological system, the estimated COD effluent concentration from the lime clarification process is 25 to 35 mg/l. Utilizing a carbon adsorption capacity of 0.5 pounds COD removed

per pound of carbon and an overall COD removal of 20 mg/l from this process, the design capacity for the advanced biological and carbon adsorption system is 350 pounds of carbon/MG. For the 100 MGD modular AWT design, the total flow (including recirculation) through the carbon adsorption process is 130 MGD.

The carbon inventory and contactor volume for this process is determined by the design wastewater surface loading and detention time. In order to comply with an NDCP effluent organic goal of 10 mg/l COD, the carbon contact time is designed for 45 minutes. The surface loading to the contactors is a typical design figure of 6 gpm/square foot. Thus, for the 130 MGD modular flow, over 0.5 million cubic feet of carbon contactor volume are required. For granular activated carbon with a density of 26 pounds/cubic foot, the carbon inventory is nearly 10,000 tons.

The carbon contactor design is based on the counter-current upflow expanded-bed flow model. [11] In using a counter-current column, fresh regenerated or make up carbon is added to the top of the contactor while the spent carbon is withdrawn from the bottom. At the design loading rate, 12 by 40 mesh carbon will expand approximately 10%. [12] This expansion of the carbon bed minimizes solids plugging problems since biological growths on the carbon will be washed through the system without washing out the granular carbon particles. For this modular system, 53 steel carbon contactors 20 feet in diameter are designed with an effective carbon bed depth of 36 feet. The bottom of the contactor is sloped at 60 degrees to facilitate spent carbon withdrawals. The total height of the contactor unit, including provisions for 10% bed expansion, is approximately 60 feet. All carbon adsorption facilities are enclosed by a building 70 feet tall. Of the 53 carbon contactors, 48 are designed to be in operation while the remaining five are used for backup and carbon-makeup storage facilities.

The spent carbon is withdrawn while the contactor is in service. It is designed so that 5% of the contents of one contactor (19,000 pounds) are removed at one time and replaced with regenerated carbon. The spent carbon slurry proceeds to drain tanks for a detention time of 15 minutes. The 50% solids slurry is then fed by a screw conveyor to the top of the carbon regeneration furnace. For the physical-chemical system, 700 pounds of carbon are spent per MG of wastewater treated. Thus for the 130 MGD flow, over 90,000 pounds of carbon will be regenerated per day. The regenerated system consists of

three multiple hearth furnaces 14 feet in diameter. These are gas-fired 6-hearth furnaces with temperatures ranging from 800°F to over 1600°F in the various hearths. In the furnace, the heat vaporizes and subsequently burns off the adsorbed carbon impurities into gaseous oxidation products and ash. The exhaust gases are wet-scrubbed to control air pollution. The turn-down ratio on the regeneration furnaces is 2:1 so that two furnaces (60,000 pounds/day capacity/unit) may handle the regeneration requirements while one is being serviced. The design fuel requirements for the regenerator furnace is estimated at 4,000 BTU/pound of carbon regenerated. This results in a natural gas (1,000 BTU/cubic foot) requirement of 0.036 million cubic feet/day for the 130 MGD modular design flow. The regenerated carbon is discharged from the bottom of the furnace to quench tanks where water is added for cooling purposes. From the quench tanks, the carbon slurry is pumped to carbon storage and washing tanks where the fines are removed prior to recycle back to the carbon adsorption system. The makeup carbon requirement is based on a carbon attrition of 7.5%. 13/ For the physical-chemical system, this results in a carbon makeup requirement of 3.4 tons per day for the modular designed plant.

For the advanced biological system, 350 pounds of carbon are exhausted per MG of wastewater treated. This amounts to a regeneration requirement in excess of 45,000 pounds of carbon/day for the modular AWT design. This is accomplished through the use of three multiple hearth furnaces 10 feet in diameter. The daily solids loading to these units is designed at 15,000 pounds/day with turn-down ratios of 2:1 thereby increasing their capacity to 30,000 pounds/day. The regeneration fuel and carbon makeup requirements for this system are one-half that of the physical-chemical system, or 0.18 million cubic feet of natural gas and 1.7 tons of granular carbon/day, for the modular design.

The total recirculated flow from the carbon adsorption and regeneration process totals 20% of the influent flow. For the 100 MGD modular AWT plant, this results in a 20 MGD flow, of which half is accounted for by blow down flows from the wet scrubbler of the multiple hearth furnace. The remaining 10 MGD originates from backwash provisions for the carbon contactors. This backwash is designed to prevent solids-plugging problems in the contactor if excessive biological growths occur. These recirculated flows, together with backwash water from the mixed-media filtration process, are sent to the influent

lime clarification process. The total recirculated flow through the carbon process is 30 MGD with 10 MGD accounted for by the mixed-media filter backwash flows.

   Clinoptilolite ion exchange. The control of nitrogen in the physical-chemical system is accomplished through the use of an ion exchange process. In this design the natural inorganic zeolite, clinoptilolite, is utilized for the selective ion exchange of ammonia nitrogen. The spent clinoptilolite is regenerated in a lime slurry. The hydroxyl ions provided by the lime react with the ammonium ions on the clinoptilolite to yield an alkaline ammonia solution which may then be air stripped for ammonia removal. This allows recycle of the regenerant; therefore, no blowdown or recirculated flows are attributed to this process.

   The basis of design for this ion exchange system is an ammonia nitrogen content of 0.5 mg/l. This is accomplished through a service cycle of 150 bed volumes at a loading of 6 bed-volumes/hour or 6 gpm/square foot. $\underline{14/}$ The ion exchange beds operate in a down-flow parallel operation. The column with a bed diameter of 20 feet, is 8 feet deep resulting in a bed-volume equal to 2,500 cubic feet. The ion exchange beds are similar to the carbon columns utilizing steel construction. For the 110-MGD modular flow design, 39 beds are necessary for continuous operation. After 150 bed volumes or approximately one day based on our design loading rate, the ion exchange beds will come off stream for regeneration of the zeolite material. This elutrient cycle is accomplished in an eight hour period. Thus the system requires an additional 13 ion exchange beds for the system which will be regenerating while the remaining 39 are in operation. The total clinoptilolite inventory for the 110 MGD modular design is in excess of 3,200 tons.

   The elutrient cycle is designed for an elution rate of 10 bed-volumes/hour or 3,140 gpm. The elution cycle is a two phase process. In the first phase, elution waters (ammonia content of 100 mg/l) from a previous regeneration are recirculated through the spent clinoptilolite columns until the ammonia in the elution water reaches some 600 mg/l. In the second phase, elution water with an ammonia content of 10 mg/l is recirculated through the spent beds until the ammonia is increased to 100 mg/l concentration. The first phase elutrient solution, with the high ammonia content, is sent to the air stripping towers. The elution volume is 4 bed-volumes/phase or 75,000

gallons/phase. During the elution cycle, the pH of the water must be maintained near 11. Also the regeneration of clinoptilolite is improved in the presence of sodium ions. Therefore, for this modular design, chemical additions to the regenerant solution include 480 pounds of lime/MG of wastewater treated together with 630 pounds of 0.2 N sodium chloride solution. For the 110-MGD design flow, this results in a daily chemical requirement of 26 tons of lime and 35 tons of sodium chloride. A lime slaker 6 feet in diameter is necessary for this process, together with a flash mixing basin with a nominal 15-minute detention time, for mixing the chemical solutions to the elution waters. Storage tanks with a total volume of nearly two million gallons are also necessary for these elution processes as the cycle proceeds from the first phase to the second to the stripping tower.

The ammonia stripping tower is designed for removing the ammonia from the regenerant solution in an 8-hour service cycle. For the 110-MGD modular design flow, nearly 3 MGD of elution water will be processed through the ammonia stripping towers. The ammonia stripping utilizes two passes of the high ammonia content solution with an ammonial removal efficiency of 85% per pass. The design hydraulic loading to the tower is 4.0 gpm/square foot. For a stripping tower flow of 4,000 gpm and a blower requirement of 300 cubic feet of air/gallon of elution water, some 1,000 square feet of tower packing area and 1.2 million cubic feet/minute of air are required for this modular design. The tower includes an open packing construction design to minimize pressure losses. The estimated pressure drop for the packed tower is 1 psi. The design temperature for optimum elution and stripping rates is approximately 75°F. [15] The maintenance of this design temperature is accomplished through the use of hot water heat exchangers and gas fired boilers. The stripping towers also are equipped with recycle regenerant basins. The ammonia laden gas from the stripping tower is then combined with the fluidizing gas entering the fluidized-bed recalcination unit where it is both partially oxidized to NO_x gas which enters the stack as a pollutant and partially oxidized to nitrogen gas, thereby entering the stack as a non-pollutant.

A design sludge generation of 400 pounds of solids/MG of wastewater treated is utilized for the ion exhcnage process. This sludge, mainly in the form of calcium carbonate, will accumulate in the stripping tower recycle basins, the chemical mixing basins and the elution storage tanks. For the 110-MGD modular design flow, approximately 22 tons of dry solids/day are collected and pumped at 10% solids by a 100 gpm capacity sludge pump to the lime recalcination process.

Mixed-media filtration. Both treatment plant systems utilize mixed-media filtration for final solids removal in order to accomplish the NDCP effluent goal of 1 mg/l of suspended solids. This is necessitated since the upflow expanded-bed carbon contactors will be partially self-cleaning with respect to solids generated in the process. Mixed-media material is composed of coarse particles (coal) on the top, medium particles (silica sand) in the middle and fine, but heavy, particles such as garnet on the bottom. In this manner, a uniform decrease in pore space with filter depth results in greater utilization of the entire filter bed, hence, longer filter runs or greater loadings when compared to other forms of filtration. For the modular AWT design, the total flow through the filters is 110 MGD. This includes 10 MGD of backwash water which has been recirculated to the lime clarification process. The design surface loading is equal to 5 gpm/sqaure foot. In order to accommodate the 110 MGD design flow, 14 concrete gravity flow type filter units are utilized.[15] [16] The dimensions of these filter units are 25 feet wide, 40 feet long and 20 feet deep resulting in a total surface area of 14000 square feet.

As previously mentioned, the filter backwash is designed to equal 10% of the filter throughput. After a filter head loss of 15 feet has been reached the filters are backwashed at a rate of 20 gpm/square foot for periods of 10 to 15 minutes. The filter run is dependent, to a great extent, on the biological activity in the carbon columns. For this design, filter runs are programmed to be in the range of 12 to 24 hours. The backwash pumps of the vertical wet pit design are rated approximately 20,000 gpm at 25-30 feet of head.

As a filter aid, alum and polymers are added to the influent stream to enhance floc formation and suspended solids removal. The design chemical dosages are alum at 20 mg/l and polymer at 0.1 mg/l. [13]

Chlorination. Chlorination is the disinfection unit process which is common to all AWT systems. The purpose of chlorination is to yield a bacteriologically safe water as defined by public health standards. However, not all virus and other resistant pathogens are inactivated under these standards. Chlorination is accomplished through the use of standard techniques presently utilized in the sanitary engineering field today. The chlorine dosage is designed at 4 mg/l which is equivalent to some 1.7 tons/day for the 100-MGD modular design flow. A typical chlorine contact time of 15 minutes is utilized. Chlorination is accomplished by means of 2-4,000 pounds/

day capacity chlorinators (1 standby) and the necessary evaporators, controls, and feed systems. Chlorine storage is designed for 15 days or over 255 tons of chlorine.

Post aeration. Post aeration is also a unit process which is common to all AWT systems. In the treatment plant systems, air is added to the renovated wastewater to insure a dissolved oxygen content of 6 mg/l prior to discharge to the receiving stream.

The aeration is accomplished by means of surface mechanical aerators. Based on a clear water aerator transfer rate of 2.85 pounds oxygen/horsepower-hour, [17] a design temperature of 70°F, α and β wastewater oxygenation factors equal to 1.0 and a residual dissolved oxygen content of 6 mg/l, the aeration energy requirements are some 50 horsepower-hours/MG. For the 100-MGD modular design, the aeration is accomplished by means of 20 mechanical surface aerators of 150 horsepower in concrete tanks with a total volume of some one million cubic feet. This corresponds to a wastewater detention time of 1.7 hours. The process layout includes four concrete tanks 100 feet wide, 200 feet long and 12 feet deep.

Treatment Plant System Layouts

Presented in Figures B-IV-A-2 through B-IV-A-4 are typical section views of key treatment components which comprise the above mentioned treatment plant systems. It should be noted that with present sanitary engineering technology, existing similar process units can be susbstituted, for the treatment elements used to meet the NDCP performance goals. Presented in Figure B-IV-A-5 is a schematic layout of the 100-MGD modular conventional activated sludge plant. Figure B-IV-A-6 presents the advanced treatment component layout for the 100-MGD modular advanced biological plant. This figure, together with the previous figure, comprise a complete advanaced biological system. Finally the physical-chemical treatment plant layout for the 100-MGD module is presented in Figure B-IV-A-7.

LAND TREATMENT SYSTEM

Treatment Capacity And Storage

Unlike the previously mentioned treatment plant systems, the land treatment system possesses an inherent capability to deal with

variable flow rates. The reason for this is that most of the land system components are in themselves storage units capable of flow regulation. Furthermore the basic components of the proposed land treatment system do not require recirculation or regeneration. The main wastewater lift station and grit removal facilities are designed for peak flow conditions. For the entire C-SELM area, the peak flows are equal to 1.5 times the average dry weather flow or 1.3 times the average daily flow throughout the year including stormwater contributions. The biologic treatment cells, or aerated lagoons, are designed for average wet flow conditions. The storage facilities at the land treatment site provide system flexibility and flow regulation for the irrigation and drainage systems. Thus the irrigation and drainage systems are not influenced by peak wastewater flows.

Land Treatment Component Design Parameters

Introduction. The land treatment system is the third AWT technology studied in this report which is designed to meet the NDCP effluent goals. This system includes wastewater lift stations which convey wastewater from land conveyance tunnels to degritting facilities and biological treatment lagoons. The effluent from these aerated lagoons is then discharged to storage facilities when irrigation of the wastewater is not feasible. The storage lagoon water is chlorinated prior to irrigation on the land at controlled rates to coincide with the critical nutrient requirements of agricultural crops during the growing season. Following advanced treatment provided by the soil medium, the percolated water is collected by a drainage system for conveyance and discharge back to the C-SELM service area.

The land treatment modular design is based on provisions for a 5,000-acre surface water storage lagoon. A 265-MGD average daily wastewater flow is the modular design flow which can be handled by such a lagoon. The lagoon size is determined so that the system may provide the necessary cooling surface for electric power generating facilities. A detailed discussion of storage lagoon and power generation design is presented in the synergism section of Appendix B, Section IV-H. The land treatment components are presented in the following section including pertinent design parameters for each process.
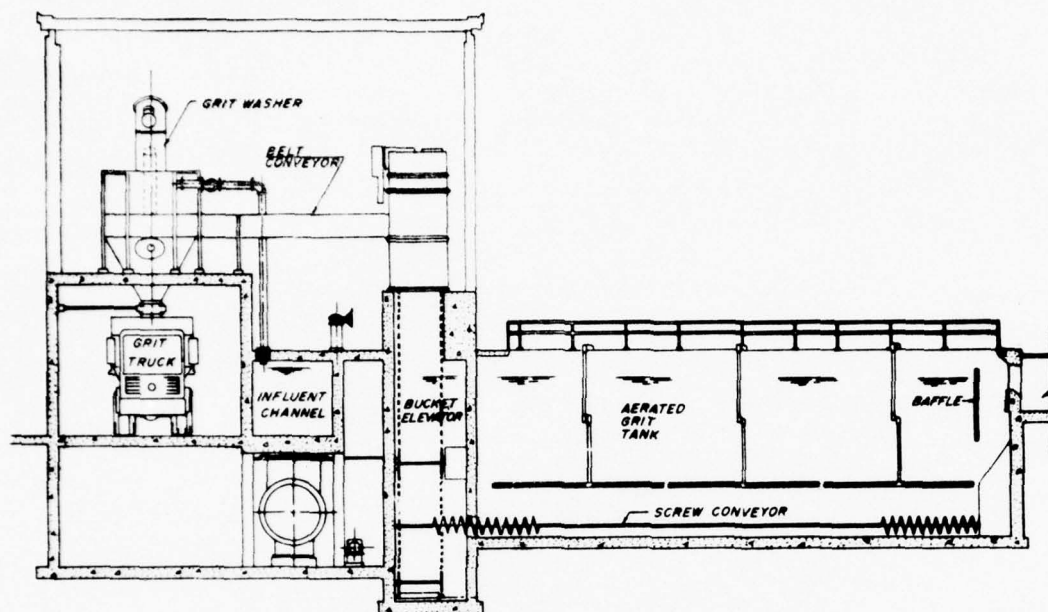
AERATED GRIT TANK

PRIMARY SETTLING TANK

PASS NO 4   PASS NO 3   PASS NO 2   PASS NO 1

AIR DIFFUSER

SETTLED SEWAGE
INFLUENT PIPE

AERATION TANK

FINAL
EFFLUENT

MIXED
LIQUID
INFLUENT

SLUDGE COLLECTOR

RETURN ACTIVATED SLUDGE &
WASTE ACTIVATED SLUDGE
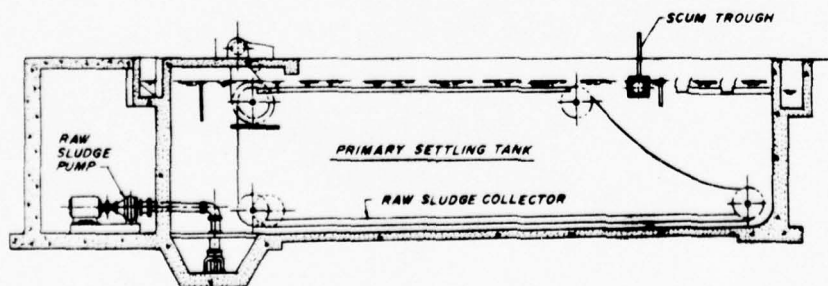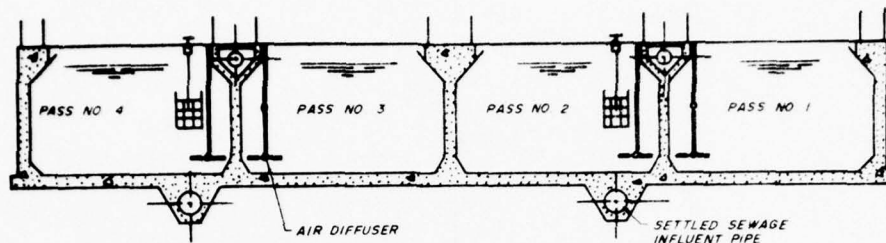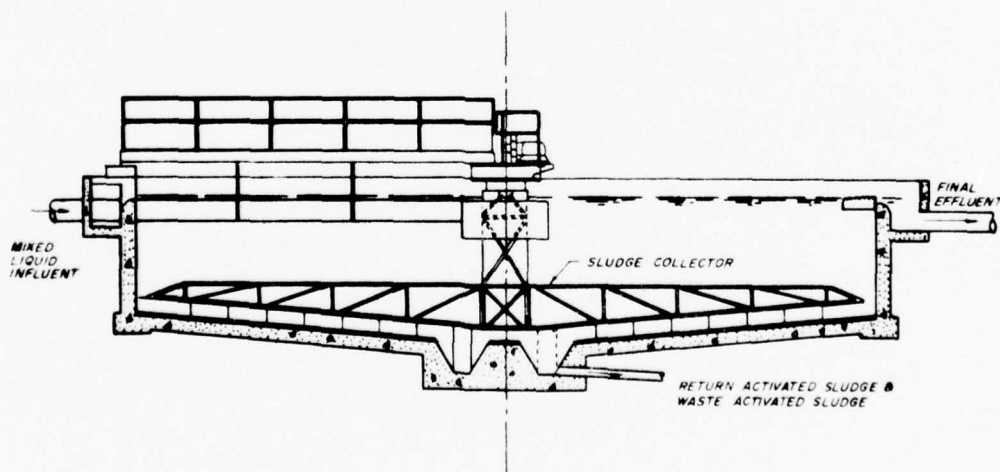
FINAL SETTLING TANK

Figure B-IV-A-2

TYPICAL CONVENTIONAL TREATMENT
PROCESS UNITS

B-IV-A-25
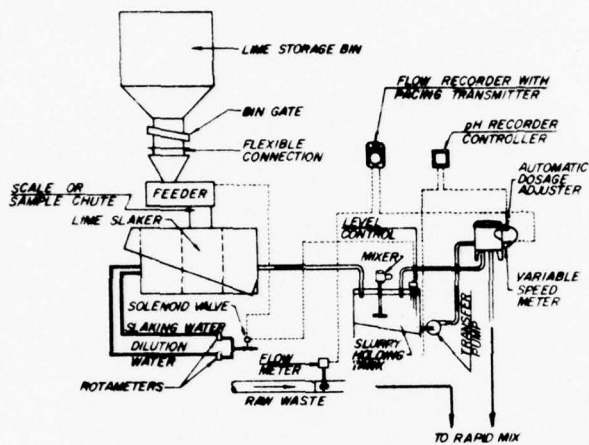
SCHEMATIC LIME FEED SYSTEM
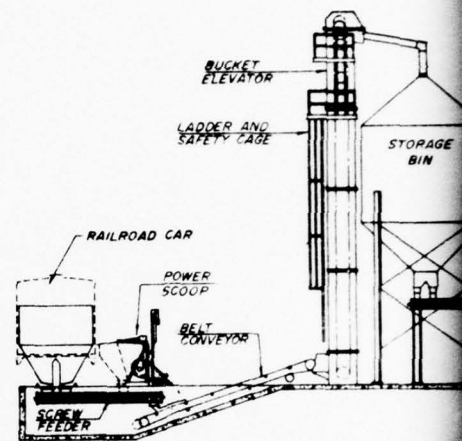
LIME STORAGE
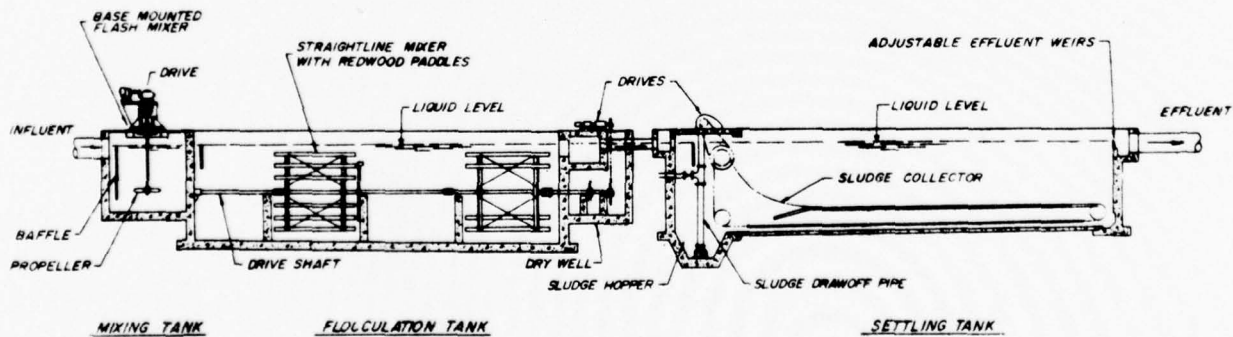
RAPID MIX — CHEMICAL FLOCCULATION & CLARIFICATION
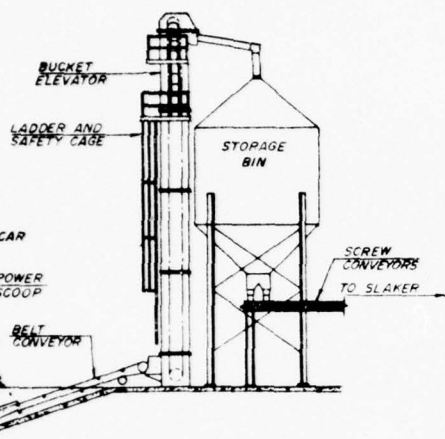
LIME STORAGE BIN
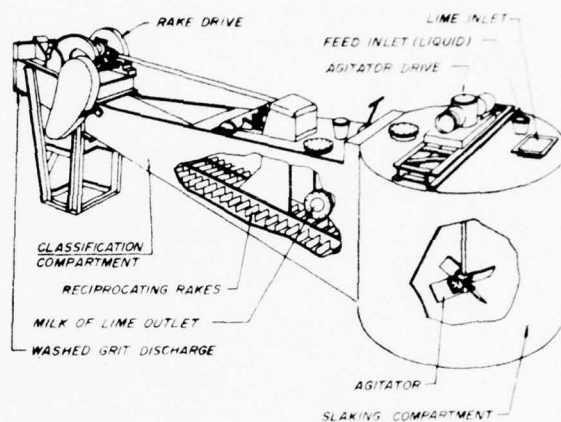
LIME SLAKER
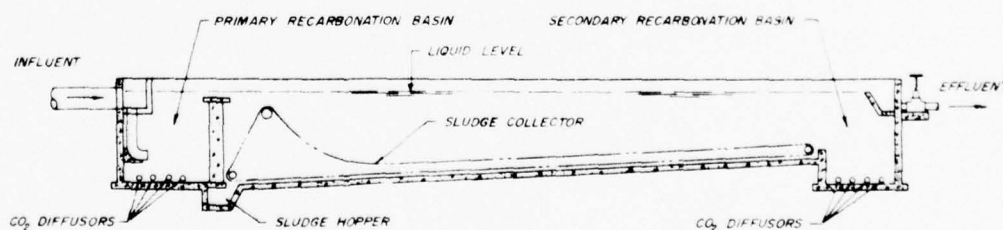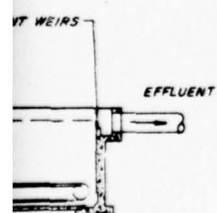
RECARBONATION

Figure B-IV-A-3

TYPICAL LIME
CLARIFICATION
COMPONENTS
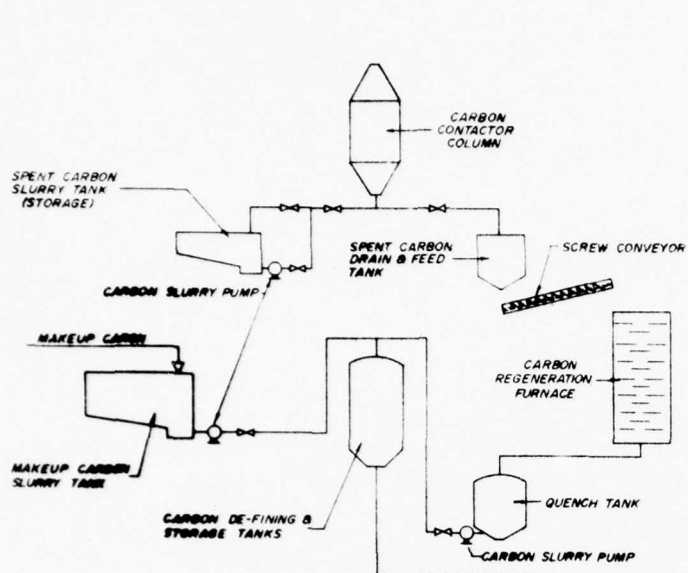
CARBON ABSORPTION & REGENERATION PROCESS FLOW DIAGRAM

CARBON CONTACTOR COLUMN

SPENT CARBON SLURRY TANK (STORAGE)

SPENT CARBON DRAIN & FEED TANK

SCREW CONVEYOR

CARBON SLURRY PUMP

MAKEUP CARBON

CARBON REGENERATION FURNACE

MAKEUP CARBON SLURRY TANK

CARBON DE-FINING & STORAGE TANKS

QUENCH TANK

CARBON SLURRY PUMP

EXPANDED BED CARBON CONTACTOR

CARBON INFLOW

SURFACE OF CARBON

OUTLET SCREENS

INLET SCREENS

OUTFLOW

INFLOW

PRESSURE VESSEL

TANK SUPPORT LEG

TANGENTIAL NOZZLES

CARBON OUTFLOW

ION EXCHANGE ELUTION PROCESS FLOW DIAGRAM

REGENERANT SOLUTION TO SPENT CLINOPTILOLITE BEDS

REGENERANT SOLUTION FROM REGENERANTED CLINOPTILOLITE BEDS

LIME & SALT FEEDERS

1   2   3
REGENERANT STORAGE TANKS

MIXING BASIN

TOWER EFFLUENT LINE

AMMONIA STRIPPING TOWER

TOWER STORAGE TANK

TOWER INFLUENT LINE

TOWER RECYCLE LINE FOR TWO PHASE OPERATION

⊗ INDICATES CLOSED VALVE
× INDICATES OPEN VALVE

NOTE: AMMONIA FROM REGENERANT TANK No I IS BEING AIR STRIPPED REGENERANT TANKS No 2 AND No 3 ARE IN REGENERATION CYCLE

CARBON INFLOW

SURFACE OF CARBON

OUTLET SCREENS

PRESSURE VESSEL

TANGENTIAL NOZZLES

CARBON OUTFLOW

DED BED CARBON CONTACTOR

CARBON IN   GAS OUT

RABBLE ARM

RABBLE TEETH

CARBON OUT

MULTIPLE HEARTH CARBON
REGENERATION FURNACE

REGENERANT SOLUTION
FROM REGENERANTED
CLINOPTILOLITE BEDS

CATES CLOSED VALVE
CATES OPEN VALVE

M REGENERANT TANK No I
STRIPPED
TANKS No 2 AND No 3
ERATION CYCLE

TO LIME
CLARIFICATION
INFLUENT

CARBON ABSORPTION
OR ION EXCHANGE
EFFLUENT

MIXED
MEDIA
FILTER

SURFACE WASH PUMP

FILTERED
EFFLUENT TO
CHLORINATION

CLEARWELL

——— NORMAL FLOW
----- BACKWASH

BACKWASH PUMP

MIXED-MEDIA FILTRATION
PROCESS FLOW DIAGRAM

Figure B-IV-A-4
MISCELLANEOUS AWT
COMPONENTS AND FLOW DIAGRAMS

B-IV-A-27

SECONDARY TREATMENT FACILITIES

CHLORINE BLDG & STORAGE

ADVANCED BIOLOGICAL PLANT

OR TO

5 - 150' DIA. DIGESTERS

KS = 120' DIA - 12 UNITS

AERATION TANKS 12 UNITS - 100' x 200'

SLUDGE STORAGE & DEWATERING LAGOONS

MARY SETTLING UNITS - 40' x 125'

REPAIR & MAINT BLDG

APPROX. 150' MIN BUFFER

AERATED GRIT TANKS

ADMIN. BLDG
LABORATORY
OWER & LOCKERS
WET WELL
BAR SCREENS

INFLUENT
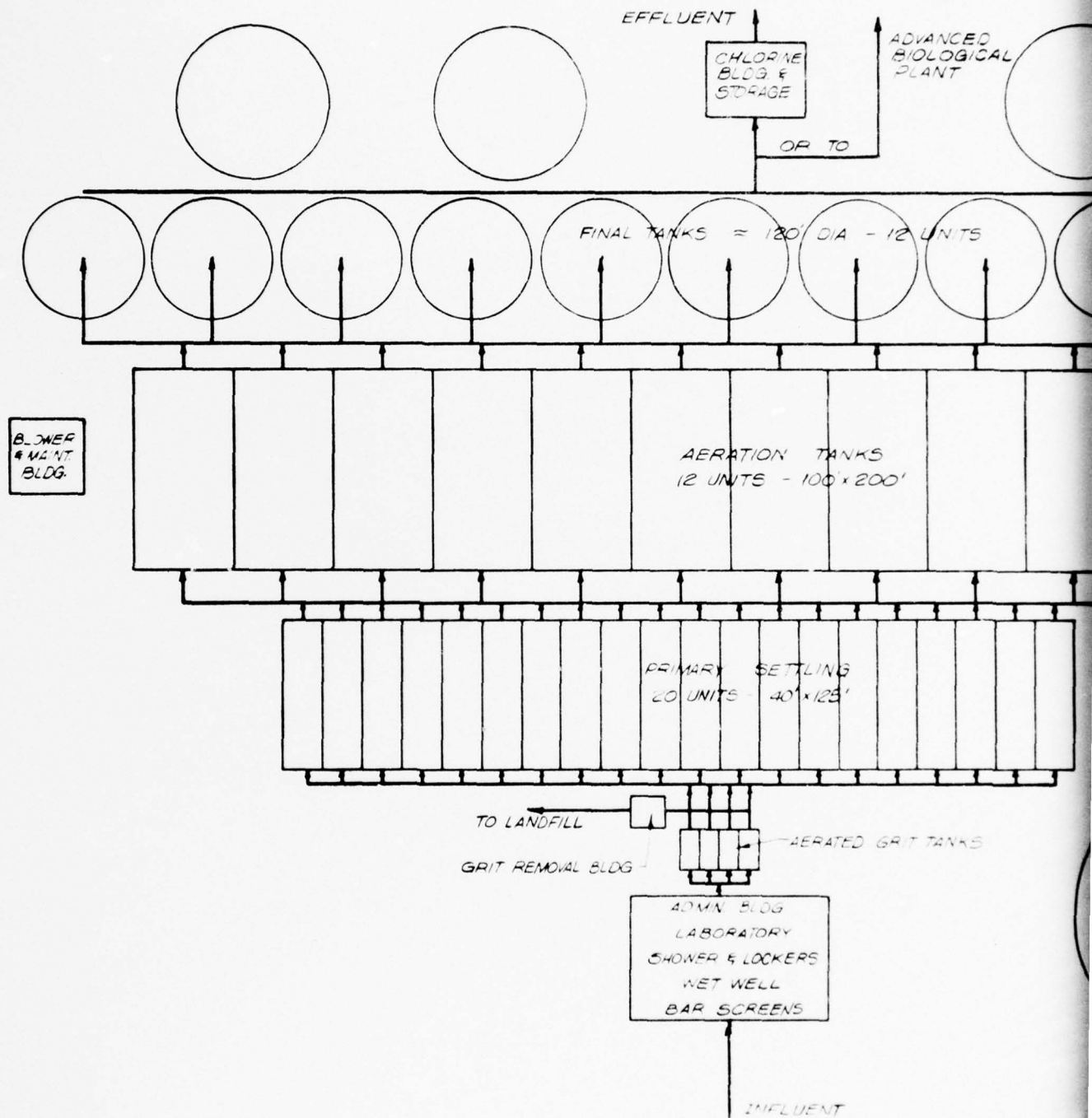
TREATMENT FACILITIES
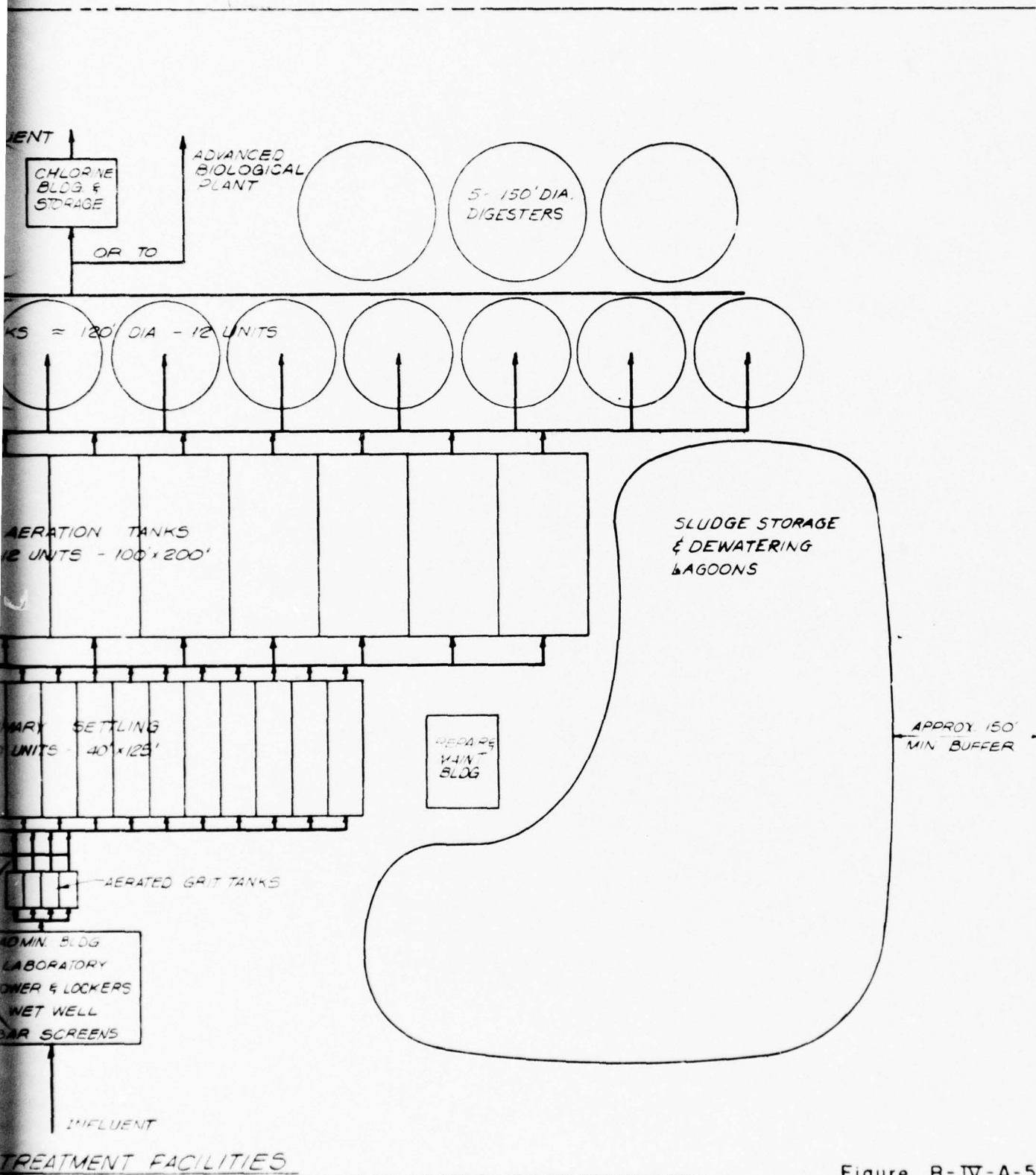
Figure B-IV-A-5

MODULAR LAYOUT FOR A
100 MGD CONVENTIONAL
SECONDARY TREATMENT PLANT

B-IV-A-28

CARBON ADSORPTION &
REGENERATION BLDG.

MULTI-MEDIA
FILTRATION BLDG

CHLORINE
BLDG

4-220' DIA RECARBONATION
REACTOR-CLARIFIERS

LIME FLASH MIXER

SOLIDS HANDLING BLDG

- THICKENERS
- FILTERS
- FURNACES
- CLASSIFIERS
- SLAKERS
& LIME STORAGE

2-100'×200

DENITRIFIC

MET
AND

6-100'×200

NTRIFIC

4-220' DIA
LIME REACTOR-CLARIFIERS

6-126 DIA
NITRIFICATION
SETTLING TANKS

BLOWER BLDG

INFLUENT

SECONDA

MENT P

ADVANCED BIOLOGICAL TREATMENT FACILIT

ON &
BLDG.

MULTI-MEDIA
FILTRATION BLDG.

CHLORINE
BLDG.

4 - 100' x 200'
POST AERATION
TANKS

EFFLUENT

12 - 126' DIA.
DENITRIFICATION
SETTLING TANKS

2 - 100' x 200'

DENITRIFICATION TANKS

METHANOL FEED
AND STORAGE

APPROX 150'
MIN BUFFER

6 - 100' x 200'

NITRIFICATION TANKS

BLOWER BLDG.

INFLUENT FROM
SECONDARY TREAT-
MENT FACILITIES

6 - 126' DIA
NITRIFICATION
SETTLING TANKS

GICAL TREATMENT FACILITIES

Figure B-IV-A-6

MODULAR LAYOUT FOR A
100 MGD ADVANCED BIOLOGICAL
TREATMENT PLANT

B-IV-A-29

4 - 220' DIA. LIME REACTOR - CLARIFIERS

4 - 220' DIA RECARBONATION REACTOR - CLA

APPROX. 100' MIN. BUFFER

AERATED GRIT TANKS

LIME FLASH MIXER

INFLUENT

ADMIN. BLDG.
LABORATORY
SHOWERS & LOCKERS
WET WELL
BAR SCREENS

GRIT REM.
BLDG

TO LANDFILL

SOLIDS HANDLING BLDG.
- THICKENERS
- VACUUM FILTERS
- FURNACES
- CLASSIFIERS
- SLAKERS
- LIME STORAGE

CARBON ADSORPTION &
REGENERATION BLDG

ION EXCHANGE BLDG

MULTI-
MEDIA
FILTRATION
BLDG

PHYSICAL - CHEMICAL TREATMENT FACILITIES

4- 220' DIA RECARBONATION REACTOR-CLARIFIERS

4- 100 x 200'

POST AERATION
TANKS

EFFLUENT

ON ADSORPTION &
NERATION BLDG

MULTI-
MEDIA
FILTRATION
BLDG

EXCHANGE BLDG

CHLORINE BLDG

TREATMENT FACILITIES

Figure B-IV-A-7

MODULAR LAYOUT FOR A
100 MGD PHYSICAL-CHEMICAL
TREATMENT PLANT

Main wastewater lift station and grit removal. This treatment component consists of an underground pumping facility which includes a structure, pumping equipment, power generation facilities and discharge shafts and piping to the degritting facilities located at the land surface. The capacity of these lift stations is dependent on the land treatment conveyance tunnel design. The wastewater lift, which is equivalent to the depth of the terminal point of the conveyance tunnel, is a function of the land site location and also of the geological formations in which the tunnel is constructed. An economic analysis indicates that an optimum tunnel lift station design utilizes minimum tunnel slopes and maximum tunnel sizes to minimize the lift of the pumping station. Intake conduits, bar screens, flow control gates and pressurized discharge shaft are all included in the wastewater lift station. For the land treatment design it is estimated that the wastewater lifts may range from 300 to over 800 feet dependent on the particular location. For the 265-MGD modular design (this is a wet average daily flow including stormwater) it is estimated that the wastewater lift is approximately 625 feet. The pumping station is designed for peak flow conditions which are equal to 1.5 times average daily dry weather flow and some 1.3 times average daily wet weather flow. Therefore, the modular wastewater lift station has a design pumping capacity of some 340 MGD at a static lift of over 600 feet, thereby requiring some 44,000 horsepower for the system design. The possibility of combining this pump station with a hydroelectric pumped storage generating station is also discussed in the section on synergisms, Appendix B, Section IV-H.

The pumping facilities lift the wastewater to degritting facilities at the land treatment site. The basis of design for these degritting facilities is the same as previously discussed for the treatment plant systems. The number of grit tank units, 20 feet wide, 40 feet long and 12 feet deep, is increased to 14 to handle the 340-MGD peak flow.

Aerated lagoon. The aerated lagoons, or biological treatment cells, provide organic removals to a secondary treatment performance level. These lagoons provide a three-day detention time for the wastewater, based on the 265-MGD average daily flow.[18] The working water depth in these lagoons is 15 feet and the total area required, including berms, is some 200 acres. The aerated lagoons are broken into three cells to facilitate maintenance and system flexibility. A lagoon

freeboard is designed for 5 feet, thereby necessitating a lagoon berm height of 20 feet. These lagoons are constructed with earthen berms or dikes. The inside slopes are 4:1 while outside slopes are 3:1. The top of these berms is 20 feet wide on which is constructed a 15-foot wide bituminous or gravel access road. The top half of the inside dike slope is stabilized by 6" of concrete to protect it against wave action. The bottom half of this berm is stabilized through the use of crushed rock, which is obtained from the wastewater tunnel construction. The total earthwork necessary to construct the three-celled lagoon for our modular design exceeds 3 million cubic yards. These lagoons are also equipped with concrete flumes, weirs and interconnection and flow distribution piping for flexibility in lagoon operation (parallel or series).

Aeration is provided in these lagoons by means of low speed surface mechanical aerator-mixers. The following mathematical model is used for determining aeration design requirements:

$$FTR = \frac{CWTR \ (Cdc) \ (\beta) - (Cr) \ (1.024)^{T-20} \ (\alpha)}{Csc}$$

where

FTR = the field oxygen transfer rate (must equal the BOD loading).

CWTR = clean water oxygen transfer rate for the aerator-mixer at standard conditions = 2.85 # O_2/HP-hr. [17]

Cdc = oxygen saturation concentration in clean water at design temperature and altitude = 8.59 mg/l.

β = ratio of oxygen saturation concentration in the wastewater to that of clean water = 0.95.

Cr = residual dissolved oxygen concentration during normal operation = 2 mg/l.

T = design temperature = 22°C.

α = ratio of oxygen transfer rate into wastewater to the transfer rate into clean water = 0.85.

$$C_{sc} = \text{oxygen saturation concentration in clean water at standard conditions } (T=20°C, \text{ altitude} = \text{sea level}) = 9.2 \text{ mg/l.}$$

Design Altitude = 700'

From the above design parameters, it is determined that the FTR equals 1.7 pounds of oxygen/horsepower-hour. For a BOD loading of 150 mg/l at design flow of 265 MGD, the total aeration horsepower requirements exceed 8,000. However, based on a mixing requirement of some 17,000 gpm/MG to insure complete mix conditions in the lagoon, and utilizing a typical mixer horsepower requirement of 1 HP/1,000 gpm, the total aerated lagoon horsepower requirement approximates 14,000. Therefore, the mixing requirement controls the power design and 90 aerator-mixers of 150 HP are utilized for this modular design.

Storage facilities. The aerated lagoon effluent is discharged to concrete flumes which convey the wastewater by gravity flow to the storage facilities. The storage facilities provide solids separation and storage of wastewater when irrigation is not feasible due to wet or freezing weather conditions. These lagoons are designed a four-month (winter storage) storage capacity of the 265-MGD modular design flow. This translates to a total volume of nearly 33 billion gallons of wastewater. The average water depth in these lagoons is 20 feet, thereby necessitating a total surface area (including the berms) of some 5,400 acres. For system flexibility, the lagoon is divided into four cells by the use of 30-foot-high berms. This height, in addition to the water depth, provides a seven foot freeboard and a three-foot dead storage volume for solids accumulation prior to sludge utilization on nearby agricultural lands. The storage berm slopes are the same as the previously mentioned aerated lagoon berms. A bituminous or gravel roadway 15 feet wide is constructed on top of the 20 foot wide berm. The berm interior slope stabilization is accomplished through a one foot layer of crushed rock. The estimated in-place earthwork requirements for the construction of the storage berms is 12 million cubic yards. These storage facilities also include inlet and outlet flumes and interconnection piping for system flexibility.

The lagoon site is designed to be constructed on relatively impermeable soils adjacent to the more permeable irrigation lands. By constructing the lagoons on clay type soils, lagoon seepage to the

groundwater is minimized. However, sufficient seepage will still occur to warrant the construction of a drainage ditch around the perimeter of the lagoon perpendicular to the direction of groundwater flow. This ditch is designed to be constructed deep enough to intercept groundwater flows so that possible contaminated groundwater from lagoon seepage cannot migrate out of the site. For the modular design, a 15 foot deep drainage ditch is constructed around two-thirds of the lagoon perimeter. Pumping facilities with a capacity of 45 MGD and TDH of some 40 feet are provided for draining the ditches and recycling the water back into the lagoon. The solids accumulating on the storage lagoon bottom also decrease the possibility of lagoon seepage.

For the land treatment system, all lagoon areas are purchased and cleared. Thus possible housing and highway relocations are considered in the design and costing of such facilities is included in the cost estimates.

The outlet flumes from the storage facilities discharge the water to chlorination facilities for disinfection prior to application on the land. These chlorination facilities are identical to design of the previously mentioned treatment plant systems. For the land treatment system, the chlorination facilities are designed for a capacity equal to the peak irrigation application rate. For the 265 MGD modular design, the land system requires chlorination facilities with a capacity of 615 MGD. At a chlorine dosage of 4 mg/l, this translates to a peak chemical demand of over ten tons per day.

Excessive amounts of chlorine have been shown to be toxic to plants. Even at concentration below toxic levels, chlorine can produce a decline in growth thus seriously reducing crop yields. [19] Research shows that chlorine residuals in excess of 50 mg/l are harmful to land plants, but that no injury occurs at residual chlorine concentrations below this level. [20] [21]

The chlorine dosage used in the land treatment system provides chlorine residuals far below the 50 mg/l concentration mentioned above. Chlorine residuals of 0.5 mg/l to 2.0 mg/l after 15 minutes are considered normal for proper disinfection of most treated sewage effluents. [2] [22] Dechlorination of the effluent prior to irrigation, therefore, is not a critical factor in the overall design and performance of the land treatment system, but is still worthy of con-

sideration as a factor of safety. Many methods of dechlorination are available for use including chemical oxidation, carbon adsorption, and aeration. For the modular land treatment design, the chlorinated effluent is transmitted to the irrigation pumping facilities by means of open channel flow. This method of transport allows time for natural aeration of the chlorinated effluent to take place resulting in the volatilization of some free chlorine and it's related compounds.

Irrigation system. Upon completion of chlorination, the lagoon effluent is pumped to the irrigation lands for application to the soil. The irrigation facilities consist of pumping stations and a force main transmission network which convey the water to irrigation machines for application onto the land. An important feature of the irrigation system is that the layout of the irrigation facilities is designed in such a manner that possible disruptions to the present land use are minimized. No building structures or roads are razed or relocated. Irrigation facilities are designed to be located so that clearing of forested areas is minimized. This system design results in an irrigation land utilization factor (actual land irrigated $\div$ gross land within irrigation site boundary) in the range of 35 to 60%. This factor is dependent, of course, on the present land use, soil uniformity, etc. For the modular 265-MGD design, an irrigation land utilization factor of 40% is used. Thus for every acre of land irrigated, approximately 2.5 acres of land are encompassed in the land treatment system.

The soil, or living filter, is the key element in advanced treatment of the wastewater. There are a number of important factors which influence the location of a land treatment system. Due to the large quantities of land necessary for the treatment of wastewater, the character or use of the land is an important consideration. The land treatment system is designed to be integrated with existing land use patterns so as to minimize possible disruptions. Thus, the more open, less-developed rural areas are the only feasible land type that can support a large land treatment system. The physical characteristic of the soil which pertains to its adaptability to an irrigation program is an essential design factor. This includes such soil parameters as permeability, infiltration capacity, soil uniformity, thickness of soil formation and topography. Soil chemistry is another important parameter in defining suitable irrigation lands. For example, cation exchange capacity, iron and aluminum contents indicate ability of soils to control nitrogen movement through the system and to remove phosphorus from the percolating wastewaters. A detailed

discussion concerning the methodology for selecting land treatment sites is presented in the soils section of Data Annex B, Section IV-A. In general the land treatment system design utilizes two different soil types. The first type is characteristic of sandy, very permeable soils. The second soil type is characteristic of silty loams or silty clay loams with permeabilities equal to one-fourth of the sandy type soils.

In the design of a land treatment system, the irrigation application system is integrated with an agricultural cropping program so as to insure proper system performance. The two-crop agricultural program which is utilized for the land treatment modular design, is presented in detail in an agricultural paper annexed to this report. 23/ This paper is presented in Section IV-A of Data Annex B. Also presented in this section of the data annex is a general discussion of wastewater irrigation impacts on agriculture. A simulation model for the irrigation and drainage components of the land treatment system is presented later in this section for the design storm condition.

For the particular soils studied in this report, the governing criterion for the amount of wastewater applied to the land is the mass of nitrogen applied to the soil. In order to meet the NDCP effluent standards, the amount of nitrogen applied must not exceed the amount of nitrogen utilized by the crops plus the nitrogen lost by soil denitrification processes plus the nitrogen lost through ammonia volatilization plus the NDCP allowable nitrogen content in the renovated waters. A detailed presentation of the calculations for the nitrogen and phosphorus balances within the land treatment system is also presented in Data Annex B, Section IV-A. In summary, the land system is based on a total nitrogen application of 500 pounds nitrogen/acre/year. By utilizing a two-crop program of corn and rye, the net projected crop utilization of nitrogen is 300 pounds/acre/year. The nitrogen lost by ammonia volatilization and soil denitrification processes is estimated at 150 pounds/acre/year. The remaining 50 pounds of nitrogen/acre/year is that which is allowed to remain in the renovated wastewater by the NDCP effluent standard. The estimated total nitrogen content of the storage lagoon effluent is 16.5 mg/l or 137.5 pounds of nitrogen/MG. Therefore, the design wastewater application rate is $500 \div 137.5$ or some 3.65 MG/acre/year. The 3.65 MG/acre/year is equivalent to some 134 inches of water applied per year. The irrigation season is designed for a 30-week period. Therefore the average irrigation rate is 4.5 inches/week. For the 265-MGD land system

module, the land irrigation requirement is equal to (265-MGD ÷ 3.65 MG/ Ac/Year) times 365 days/year which equals 26,500 acres. For a land utilization factor of 40%, the gross irrigation requirement for the land module equals some 66,000 acres. During the early spring and late fall, the irrigation application rate is less than the average while in the summer months, when crop growth is at a maximum, the application rate for the land treatment system is six inches of wastewater/week. Applying six inches of water for one week over the 26,500 acre irrigation land module is equivalent to a 615-MGD peak flow. Therefore, the chlorination facilities and the irrigation pumping stations and force mains are designed for a 615-MGD capacity. For the modular design, the total dynamic head associated with the irrigation distribution system is designed at approximately 300 feet. Therefore, the total horsepower requirements for the irrigation pumping stations exceeds 38,000 HP. The design velocities in the distribution system are in the range of 2 to 8 feet/second (fps). The irrigation distribution system is comprised of asbestos cement, steel or reinforced concrete force mains with sizes ranging from 16 to 84 inches in diameter. The total length of this distribution system is a function of land utilization factor for a particular site. For our modular designed system with a utilization factor of 40%, the total length of the distribution system approximates 0.85 million lineal feet.

The terminal points of the irrigation distribution system are center pivot rotating irrigation machines which apply the pretreated wastewater to the land. For the sandy type soils, which are characterized by a permeability of 400 gpd/square foot, three different size irrigation rigs are utilized. They are 1,000, 1,500 and 2,000-foot radius machines with an area coverage of 72, 162 and 288 acres, respectively. The hydraulic capacities of these irrigation machines range from 1.7 MGD to 6.7 MGD. The rig pipe which distributes the water from the machine is supported by steel trussed towers with spans ranging from 75 to 150 feet so that the weight per span is nearly constant. The water pressure at the pivot of the irrigation machine is designed at 35 psi and the pressure loss through the machine is not to exceed 15 psi. These machines are designed to be electrically gear driven with typical horsepower requirements of 25 HP/unit. The machine is approximately 8 feet above the ground so as to clear all crops. The wastewater is applied through low pressure spray nozzles directed downward with resultant large droplet size to minimize aerosol spray effects. The distribution pattern of these nozzles is in the shape of a peanut with a total spray of 40 feet long by some 20 feet wide as graphically presented in Figure B-IV-A-8.
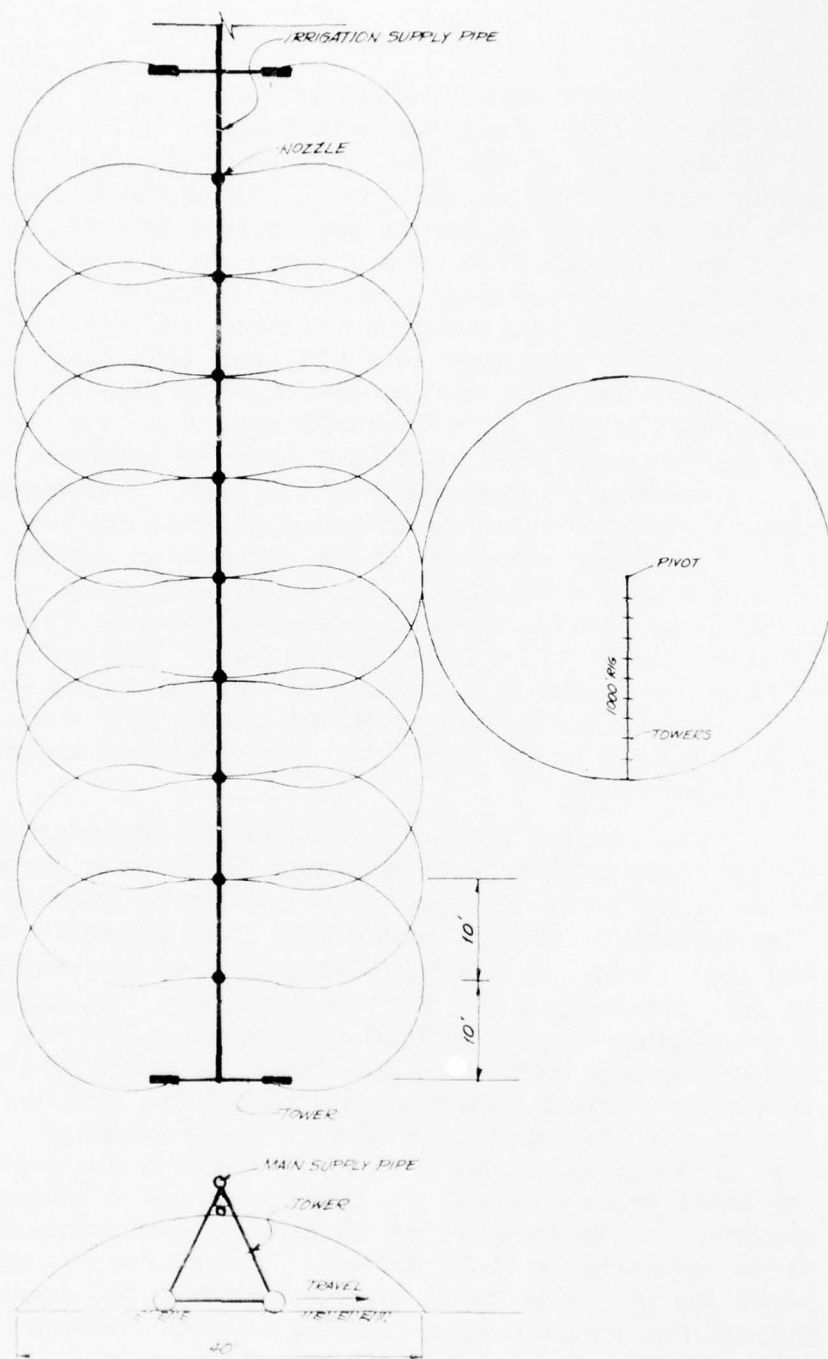
IRRIGATION SPRAY PATTERN FOR SANDY TYPE SOILS

For the other less permeable type soils (100 gpd/square foot) studied in this modular design, only 1,000-foot radius irrigation machines are utilized. This limitation on rig size is made since the infiltration capacity of these soils is exceeded by the wastewater application rate at the outer portion of the 1,500 and 2,000-foot radius machines. This is unacceptable since surface runoff would occur resulting in a nonuniform application of wastewater in low areas thereby causing overloaded conditions and reduction in system performance. To insure that the application rate from the 1,000 foot machine does not exceed the soil infiltration capacity, special outriggers are designed so that the spray pattern at the end of the machine increases some 60 feet (total pattern length of 100 feet is equal to this 60 feet plus 40 feet from the spray nozzle) and gradually decreases to the center pivot as shown on Figure B-IV-A-9.

Drainage system. After passage through the soil, or living filter, the reclaimed water is collected in a drainage network of pipes and channels to central access points for discharge to a reclaimed water tunnel system and subsequent transmission back to the C-SELM receiving streams.

The land treatment system drainage capacity is equal to the irrigation application rate of 6 inches/week or the equivalent 615 MGD for the modular designed site. The basic drainage criteria is the maintenance of a minimum aerobic soil zone five feet deep to facilitate the chemical, physical and biological soil treatment processes so that the NDCP effluent standards may be attained. Thus prolonged saturation and increased salt contents of the soils and resultant crop losses are eliminated by this drainage system.

The drainage system consists of drain tile (plastic pipe) underlying the irrigation machines. The water collected from the tile flows to gravity sewer pipe. Dependent on the particular location & topography, the gravity sewer pipe may either discharge to a surface channel, interconnect with a larger gravity sewer pipe draining an adjacent rig, or discharge to a wet well for force main transmission to a drainage channel or access point. The drainage system is carefully designed to fit the existing topographical conditions. Where possible the drainage channels are located along existing drainage ditches. The drainage system is designed so that the reclaimed water is collected at central access points. At these access points, the water is dropped through a shaft to a drainage tunnel network which services a number of land modules, for transmission back to the C-SELM service area.

Figure II-IV-A-9

IRRIGATION SPRAY PATTERN FOR SILT LOAMY SOILS

The energy expended by the water in the drop shaft will provide sufficient reaeration for the reclaimed water prior to discharge to receiving streams. The drainage system is also designed to provide adequate storage of the irrigated water so that reclaimed water reuse flows from the land treatment sites to the C-SELM receiving streams can take place during the winter when the irrigation system is not operating. A detailed analysis of the soil infiltration capacity and irrigation application rates is presented in the soil section of Data Annex B, Section IV-A

Another feature of the drainage system is the collection of surface runoff flows from the rural area within the land site boundary. This is necessary so that surface runoff flows do not contaminate or degrade the reclaimed water in the drainage channels. Surface runoff control in the land site is accomplished by gradually contoured earthen berms constructed on the land site, as shown in Figure B-IV-A-10. These berms, with heights of one or two feet, are constructed perpendicular to the direction of the runoff flow. For the 1,000-foot radius irrigation plot, two such berms would be adequate to retain the storm runoff from the irrigation land and from the adjoining pasture and crop lands which are not irrigated but are located upstream of the irrigation land. This retained runoff would eventually percolate through the soil and be collected by the drainage system.

The drain tile is designed to drain the design irrigation application rate of six inches/week with a surcharge head of less than one foot. For the permeable, sandy-type soils, (permeability = 400 gpd/ft^2), the tile spacing is 400 feet and is comprised of six or eight inch diameter plastic pipe. For a 1000-foot radius irrigation plot, approximately 8000 feet of six inch diameter plastic drain tile is required. For the less permeable sandy or silt loam type soils (permeability = 100 gpd/ft^2), the tile spacing is 100 feet using four-inch diameter plastic pipe. For a 1000-foot radius irrigation plot, 29,000 of four inch diameter plastic drain tile is required. The design velocities in the drain tile range from 0.5 to 1.0 fps. For the four and six-inch diameter drain tile, the slope is 0.3%. For the eight inch diameter drain tile, the slope is 0.2%. The minimum depth of the drain tile is 13 feet below the ground surface. At this depth, the drainage system can handle the 100-year storm on record and provide adquate storage for winter reuse flows without having the drainage water rise into the top 5-foot aerobic zone for a period greater than 24 hours. Thus crop damage from large storm events due to prolonged saturation of the root zone is prevented. A hydraulic analysis of the drainage system for the design storm is presented later in this section.

DRAINAGE DITCH

BERMS FOR
PREVENTING
SURFACE RUNOFF
CAUSED BY
HEAVY STORM

SUBSURFACE
DRAINAGE SYSTEM

BOUNDARY OF
IRRIGATION AREA

MAJOR ROADS

A

A

BERMS

START TIME PONDING
OF STORMWATER

SEWER PIPE

DRAIN PIPE
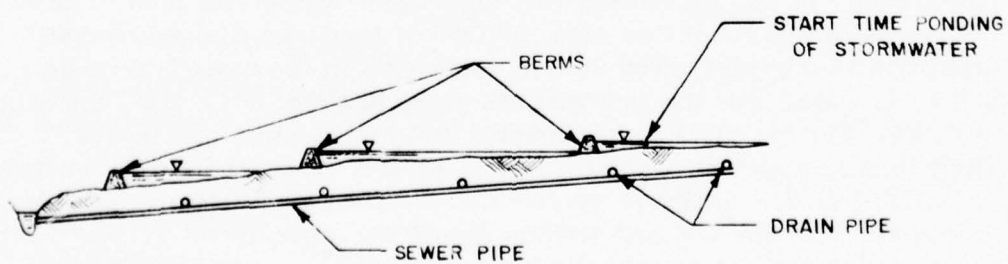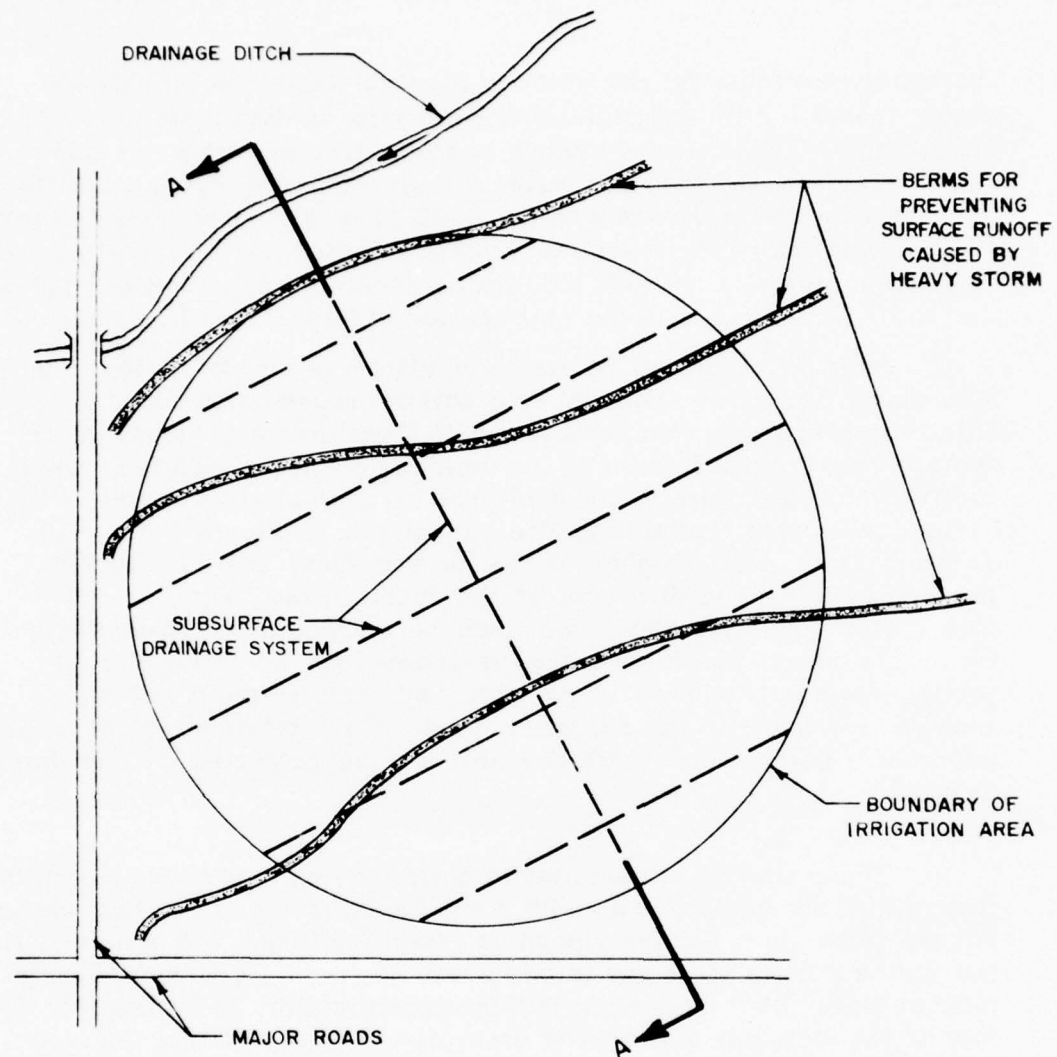
SECTION A—A

Figure B-IV-A-10
CONTOURED DRAINAGE SYSTEM

Upon collection by the drain-tile system, the reclaimed water flows to sewer pipe for drainage from the irrigation area. In order to drain a 1,000-foot radius irrigation machine, sewer pipe is required in sizes ranging from six to sixteen inches in diameter with slopes in the range of 0.08 to 0.3%. The minimum velocity in this sewer pipe is designed at 1.5 fps. A control valve is installed at the outlet of the sewer pipe draining each irrigation machine. This valve, which is utilized in late summer, provides control of the drainage outlet capacity so that storage of the reclaimed waters and their ultimate discharge during the winter months may take place. Dependent on the topography of the particular site and the location of the irrigation machines relative to each other, the sewer pipe may: (1) discharge to surface channels (2) combine with another sewer pipe system of a irrigation machine or (3) discharge into a wet well for transmission via force mains to surface channels or access points. For the most part, the reclaimed water discharges to drainage channels which are designed to conform with the existing drainage relief patterns. These channels are typically 16 feet deep with side slopes of 2:1 and top channel widths of some 70 feet. Since velocities are in the range of one to four fps, slope stabilization by the use of crushed rock is designed into the system. For the upstream drainage portion of the land module, the water depths in these channels are as low as one foot. For downstream portions of the land drainage module, the channel water depth prior to discharge to an access point may exceed eight or nine feet.

As mentioned previously in this section, the drainage channel network transmits the reclaimed water to tunnel access points. These access points are water drop-shafts whose depths are dependent on the geological formation of the particular site. In general, these access points link the surface drainage system with the tunnel drainage system at depths of 200 to 300 feet below the surface. The drainage tunnel is designed for a six inch/week capacity. These are unlined, mole-constructed tunnels which are identical in construction design to the wastewater tunnels discussed in detail in Appendix B, Section IV-E. The land treatment drainage system terminates with the drainage tunnels leaving the land site boundary. Once beyond the land site, these drainage tunnels are incorporated into the reuse system and are hence, referred to as reuse tunnels which transmit the reclaimed water back to the C-SELM service area for various reuse purposes.

For the 265-MGD modular land treatment system, approximately 2.7 million lineal feet of drain tile are utilized for draining 26,500 acres of sandy type soils while 10.6 million lineal feet of tile are necessary to drain silty or clay loam type soils of a comparable area. This drain tile discharges to some 0.7 million lineal feet of sewer pipe in sizes ranging from six to 40 inches in diameter. In certain areas where the topography is such that force main transmission of the re-claimed water is necessary, some 40,000 lineal feet of 66-inch dia-meter force main is required. Pumping requirements of some 9,000 horsepower is also necessary for this modular design. Finally, the drainage tunnel system is comprised of some 85,000 lineal feet of tunnel with 12 to 16 foot diameter and with capacities ranging from 600 to some 1,000 cfs.

Irrigation And Drainage Performance Analysis

Simulation model inputs. As mentioned previously in this sec-tion, a simulation model has been constructed to analyze the perform-ance of the irrigation and drainage system components. A statistical analysis has indicated that the rainfall recorded in July, 1957 and the year 1957, is the 100-year storm on record on a monthly and annual basis, respectively. This 1957 rainfall has, therefore, been used as the design storm for the time series input to the model.

The agricultural tillage schedule which guides the wastewater application, varies with the type of crop being planted. A detailed agricultural paper is presented in Data Annex B, Section IV-A which includes examples of tillage schedules based upon consultations with governmental and university agricultural experts. A typical no-tillage schedule has been adapted from these discussions for use in this model and is presented in Table B-IV-A-3.

The distribution of the wastewater application rate is carefully arranged so that it will closely parallel the crop nutrient needs through-out the growing season. This tailored application of nutrients permits control of groundwater nitrogen concentrations to within NDCP standards while applying yearly applications of 134 inches of pretreated waste-water. The irrigation rate schedule is also shown in Table B-IV-A-3. The application of wastewater is interruped when rainfall intensity is greater than one and one quarter inch/day. This operational design prevents undesirable overland flows.

TABLE B-IV-A-3

TYPICAL TILLAGE AND WASTEWATER APPLICATION SCHEDULE

Period Date	Tillage	Irrigation	Average Weekly Application Rate (inches)	Duration (days)
4/1 - 4/28	No	Yes	4	28
4/29- 5/11	Yes	No	0	13
5/12-6/1	No	Yes	1	21
6/2 - 6/12	No	Yes	2	11
6/13 - 6/30	No	Yes	6	18
7/1 - 7/31	No	Yes	6	31
8/1 - 8/25	No	Yes	6	25
8/26-9/17	Yes	No	0	23
9/18-9/30	No	Yes	6	13
10/1-10-30	No	Yes	6	30
10/31-11/13	No	Yes	5	14
11/14-11/19	Yes	No	0	6
11/20-11/30	No	Yes	4	11

NOTE: The above schedule reflects a bell shaped distribution of 135 inches of pretreated effluent applied as irrigation water.

The drain tile system is designed for a level of 13 feet below the ground surface to provide adequate underground water storage between the drain tile and the lower boundary of the aerated soil zone. This storage prevents intrusion of groundwater into the aerated (top five feet of the soil) and crop root (top three feet of soil) zones and also provides for a reclaimed water supply for winter reuse flows in the C-SELM streams. This storage for winter reuse flows is accomplished through the use of flow regulation valves installed at the outlet of underdrainage system for each irrigation machine. The soil impact data is that of a relatively tight silt-loam soil characterized by a permeability of 100 gpd/square foot.

Also programmed into the model is the assumption that in order to provide enough reuse water supply during the 20-day tillage period occurring in late August and during the winter season thereafter, the drainage flow regulation system begins on August 25. These valves are not reopened during the storage season unless the groundwater table in the irrigation area exceeds a level which is one foot below the aerated soil zone. These valves are closed when the groundwater table recedes to a level two feet below the aerated zone (7 feet below the ground surface).

Simulation model results. By utilizing the above-stated design conditions and operational procedures, the irrigation and drainage system simulation model provides such system performance results as application rate, drainage outflow and groundwater levels. The mathematical relationships programmed into the simulation model, together with the computer analysis results, are presented in Data Annex B, Section IV-A. These results indicate that for the design storm year, the land system model can accommodate the average yearly wastewater application of 134 inches during the design irrigation period. The drainage system can also accommodate this wastewater application together with the design storm events while preventing the intrusion of groundwater into the aerated soil zone for a period exceeding 24 hours. Thus, no crop damage due to high groundwater conditions can reasonably occur under this design. As a measure of comparison, it is estimated that for the same type, agriculturally-utilized soils which are presently drained by tile, the groundwater intrusion due to stormwater into the three-foot root zone occurs at a frequency of every five years.

The analysis also indicates that the design of the system is sufficient to provide winter reuse flows to the C-SELM study area. It should also be noted that the system performance is analyzed under stringent soil constraints. For sandy type soils, the storage coefficient is twice that programmed into the model for silt-loam type soils. Thus the storage capacity for sandy soils provides a safety factor of two for the simulation model. It is also apparent from this analysis that flood relief is not a problem because the groundwater storage design is controlled by winter reuse water supply considerations.

The computer analysis results presented in Data Annex B, Section IV-A also includes system performance data for a conventional tillage schedule as outlined in that section.

Land Treatment System Layout

Presented in Figure B-IV-A-11 are typical section views of key components which comprise the above-mentioned land treatment system. Presented in Figure B-IV-A-12 is a schematic layout of a 265-MGD modular land treatment system. For the total site boundary the estimated land use classification is presented as follows:

Land Use Classification	% of Total
Irrigation Land	34
Lagoons	8
Sludge Disposal	7
Unsuitable Soils	8
Green Space	7
Streams, Railroads and Roads	2
Dwellings and Urban Centers	4
Crop and Pasture Land	30
	100

It should be noted that the irrigation land utilization factor for the module is calculated by taking the total site area less the lagoon and sludge areas and dividing this into the irrigation land. For this module, utilization factor equals $\dfrac{.34}{1.00 - (.08 + .07)} = 0.40$ or 40%.

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5'-0 FREEBOARD

CONCRETE WAVE PROTECTION

CRUSHED ROCK LINING

20'-0

5

1

4

CLAY FILL

15'-0

7'-0
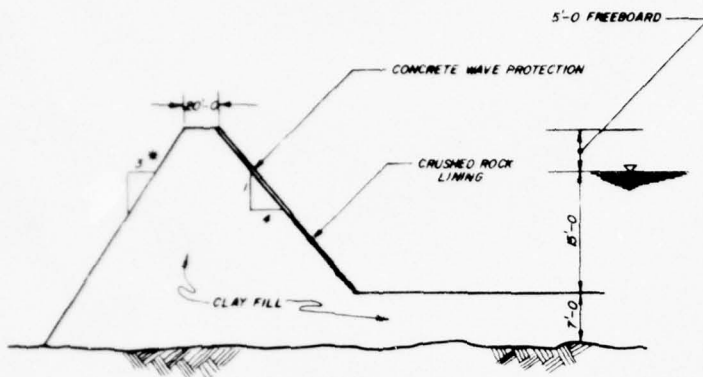
TYPICAL AERATED LAGOON BERM SECTION

AERATOR-MIXERS

5'-0 FREEBOARD

18" SQUARE COL

15'-0

1'-0
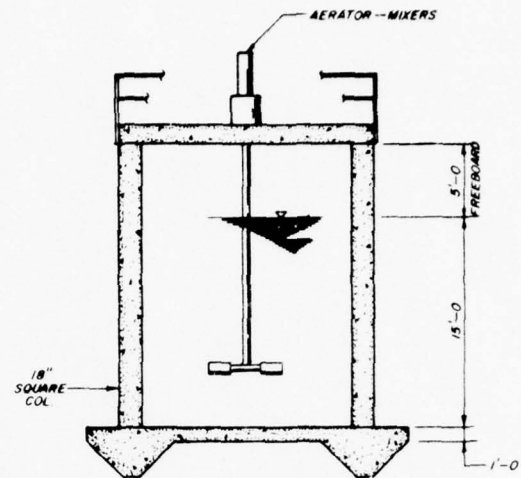
TYPICAL AERATOR-MIXER & SUPPORT SECTION

CRUSHED ROCK LINING

20'-0

7'-0 FREEBOARD

3

1

27'-0 AVERAGE CLAY FILL

1

4

20'-0 AVERAGE WATER STORAGE DEPTH

3'-0 AVERAGE SOLIDS STORAGE DEPTH
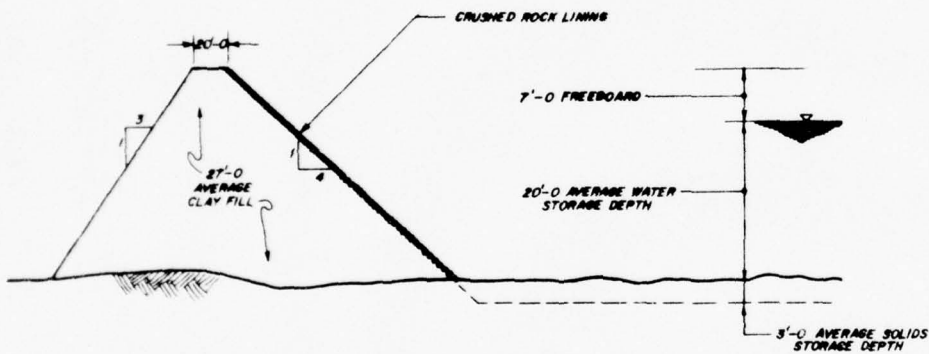
TYPICAL STORAGE LAGOON BERM SECTION

65 TO 70'

16'-0

CRUSHED ROCK SLOPE STABILIZATION

ELEVATION VARIES

16'-0

SEWER PIPE DRAINING IRRIGATION LANDS

1

2

2

6'-0
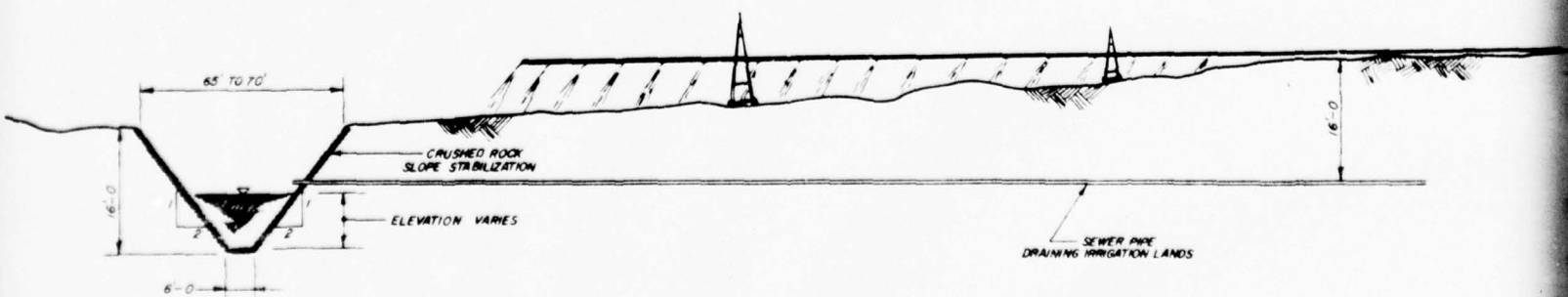
TYPICAL DRAINAGE CHANNEL SECTION

* NOTE: BERM SLOPES ARE NOT DRAWN TO SCALE
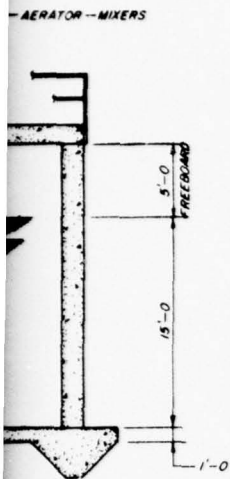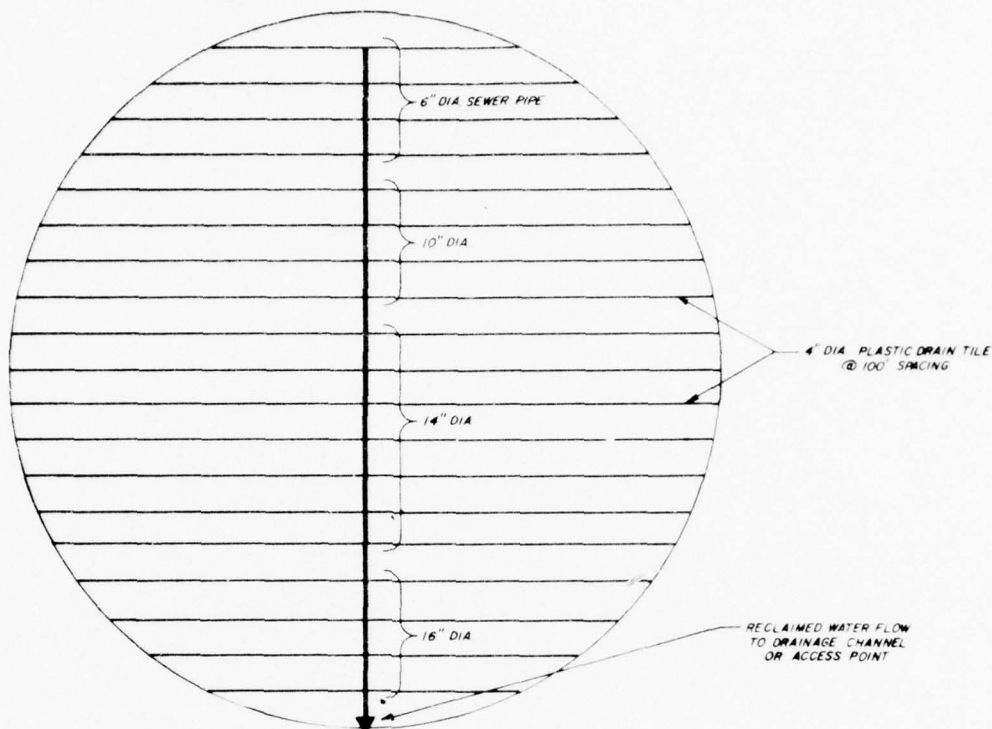
DRAINAGE SYSTEM FOR A 1000' RADIUS IRRIGATION MACHINE
ON LOAM TYPE SOILS
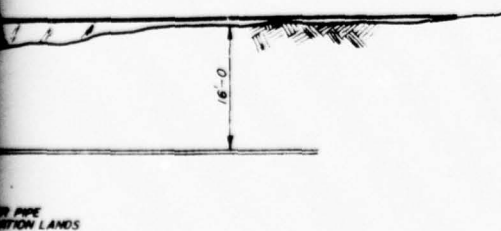
DRAINAGE SYSTEM FOR A 1000'
RADIUS IRRIGATION MACHINE
ON SANDY TYPE SOILS
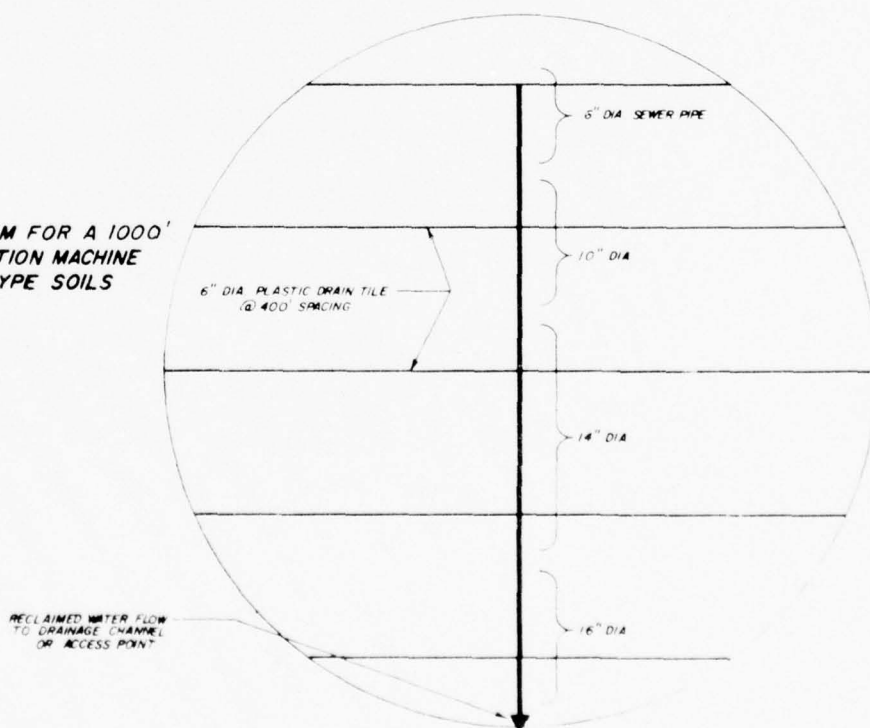
Figure B–IV–A–11
TREATMENT COMPONENTS OF LAND SYSTEM

B–IV–A–
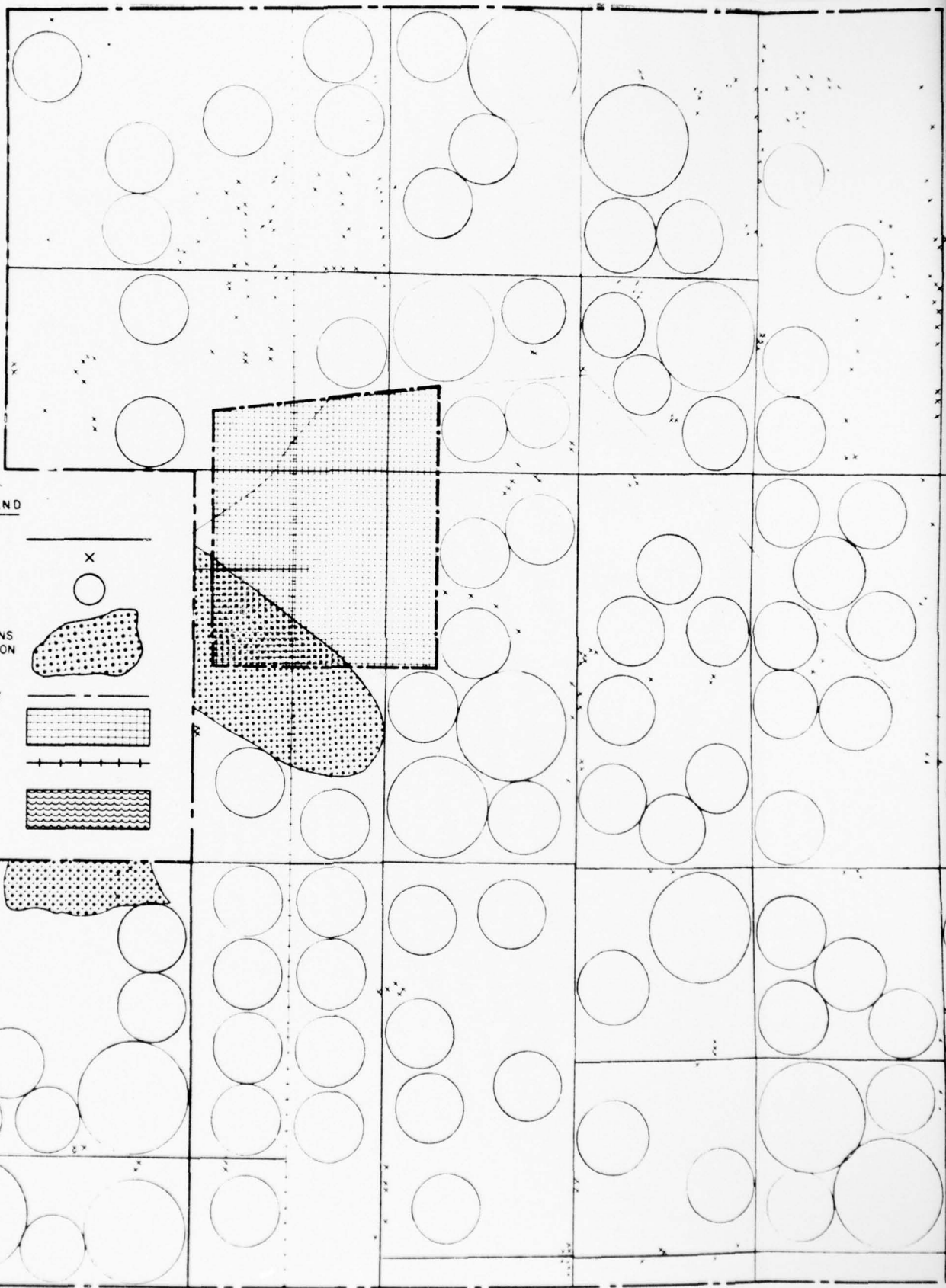
LEGEND

ROAD OR HIGHWAY
BUILDING OR STRUCTURE   ×

IRRIGATION AREA

INADEQUATE SOIL CONDITIONS
FOR WASTEWATER IRRIGATION

IRRIGATION SITE BOUNDARY

URBAN CENTER
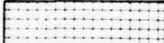
RAILROAD
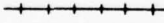
STORAGE AND AERATED
LAGOON FACILITIES

Scale   0   2000   4000 Feet
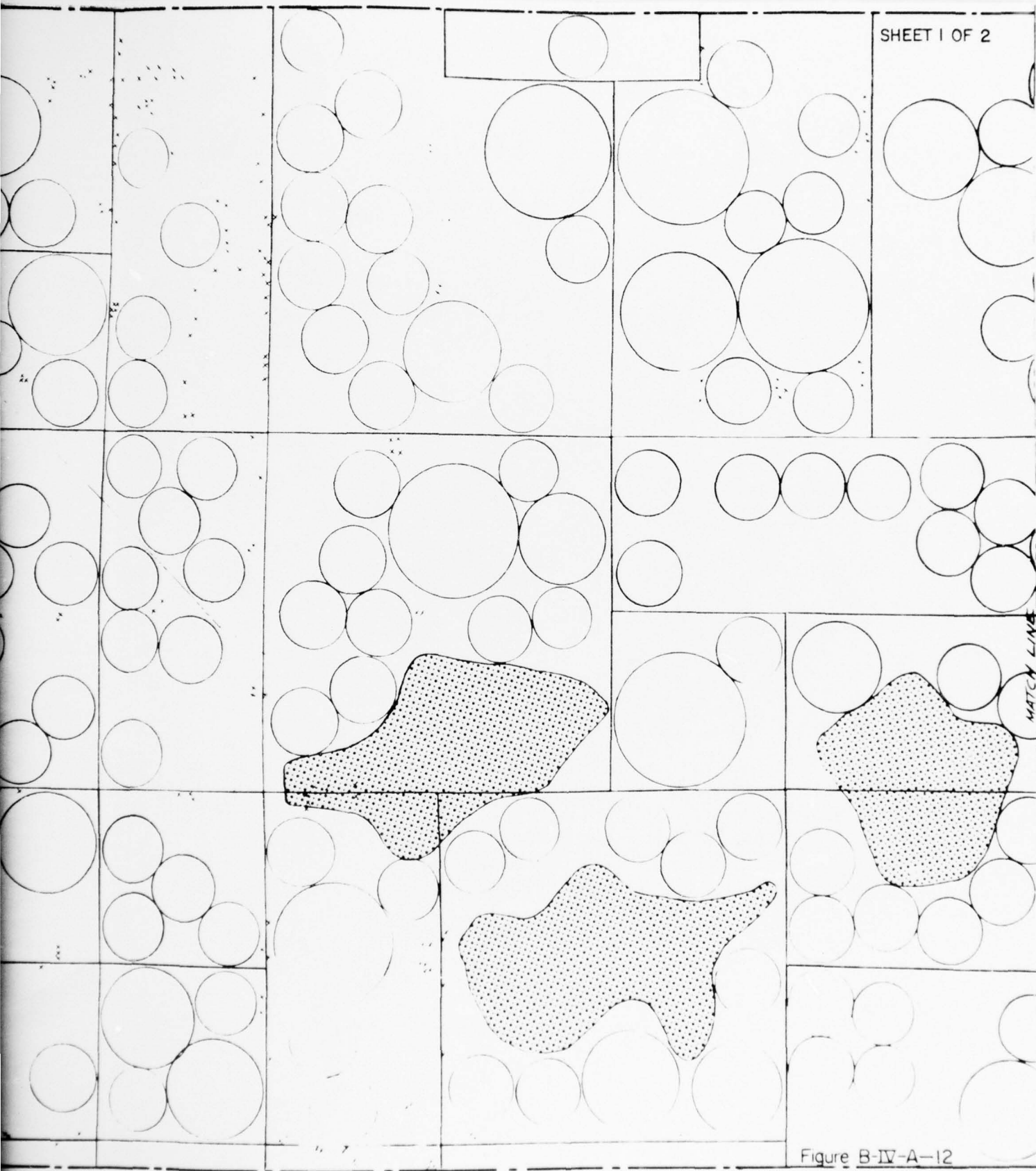
Figure B-IV-A-12

SYSTEM LAYOUT FOR A 265 MGD LAND TREATMENT MODULE

B-IV-A-48

MATCH LINE

STORAGE LAGOON CELL

AERATED LAGOON

SLUDGE DISPOSAL
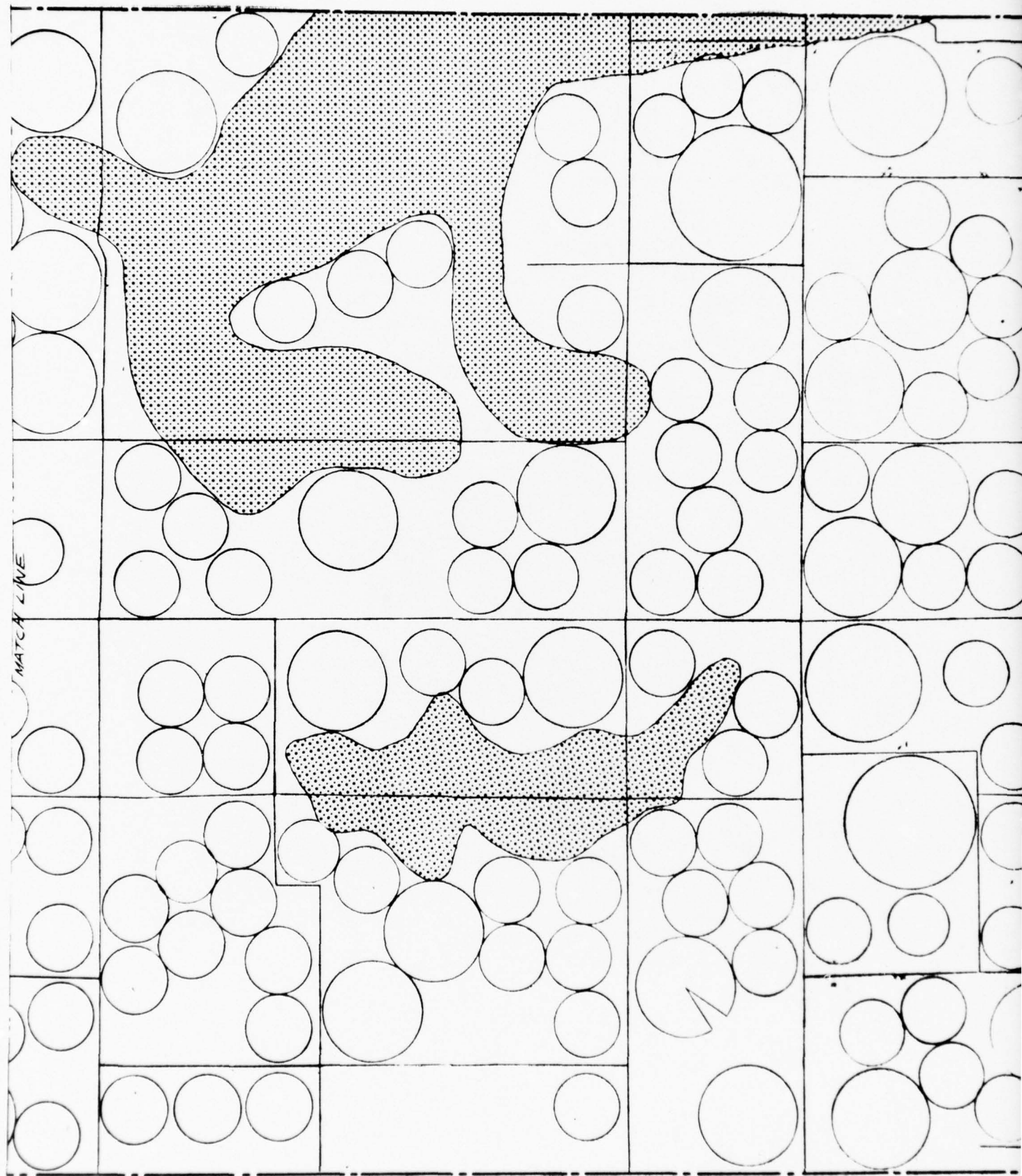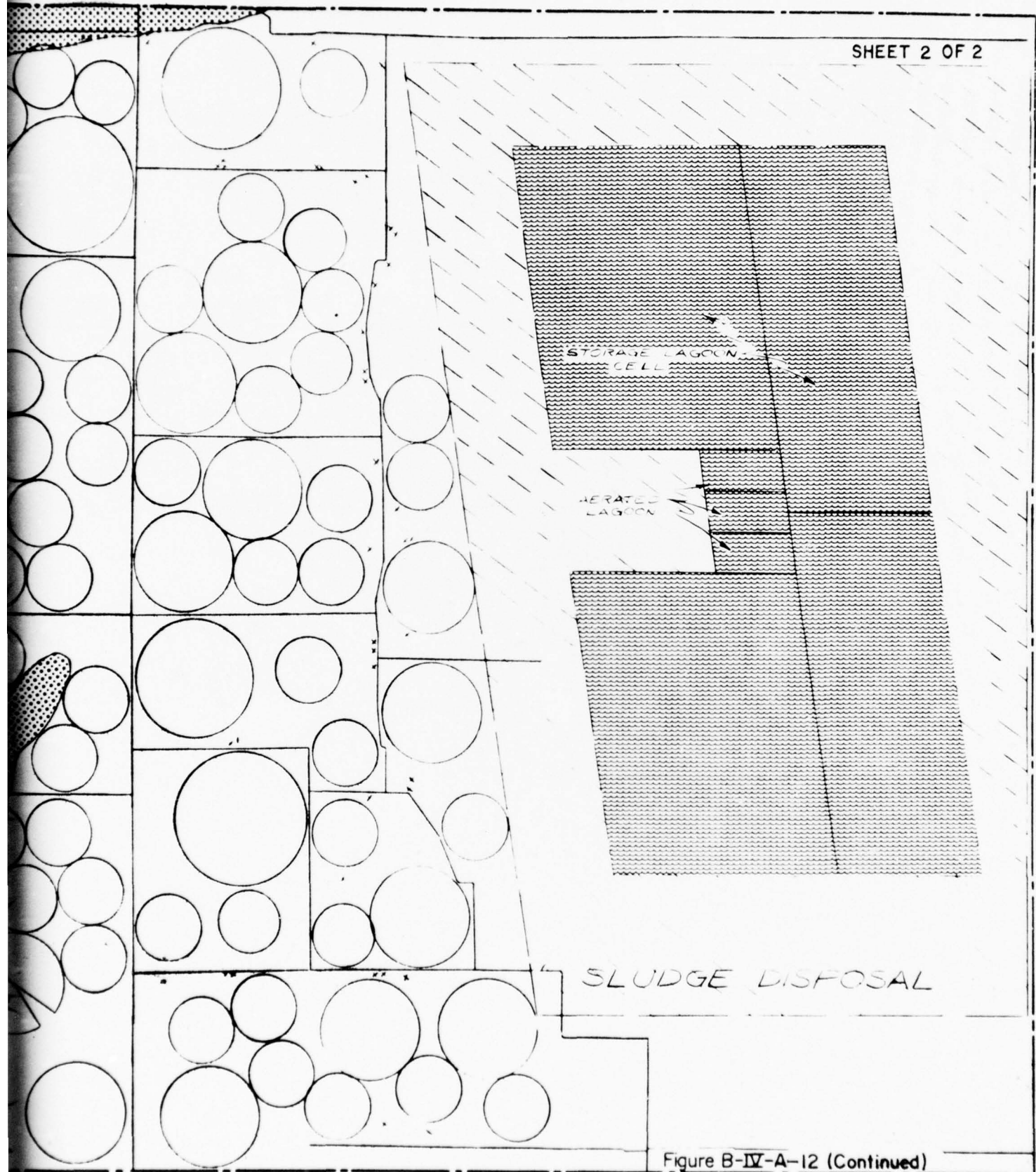
Figure B-IV-A-12 (Continued)

SYSTEM LAYOUT FOR A 265 MGD LAND TREATMENT MODULE

IV.   COMPONENT BASIS OF DESIGN

B.   INDUSTRIAL TREATMENT SYSTEMS

INTRODUCTION

C-SELM Industry

Industry in the C-SELM area can be divided into the following classifications: present industrial contributors to municipal treatment plants, present critical industry dischargers to surface waters, and present non-critical dischargers to surface waters.

Present industrial contributors to municipal treatment plants generate a wastewater acceptable to municipal collection systems either directly or by means of pretreatment.

The critical industry dischargers in the C-SELM area are, by definition, the steel and petroleum industries.   These industries are given a critical designation by virtue of their very large flow contribution to C-SELM watercourses.   Treatment is presently provided by these industries prior to wastewater discharge.

Present non-critical industry dischargers to surface waters include all other industries presently discharging effluent to surface waters.

Purpose

The purpose of this sub-section is to identify the management systems associated with C-SELM industrial wastewater treatment so that the impact of NDCP standards on industrial treatment strategies can be determined.   This will be accomplished by first discussing some general management concepts followed by a more detailed description of industrial treatment for some special industrial classifications. C-SELM industrial flows are presented in Appendix B, Section III-B. The cost implications associated with NDCP standards for the C-SELM area will be discussed in Appendix B, Section VI-B by industrial classification.

General Management Concepts

Recycle. Many industries generate wastewater with quality characteristics perfectly adequate for recycling for further industrial use. Sometimes minimal pretreatment is required for recycling. Recycling accomplishes two goals: first, it minimizes demands on an often over-taxed water resource; second, it minimizes the quantity of wastewater that will ultimately be rejected or blowndown for treatment prior to discharge to a receiving water. As treatment requirements, and thus treatment costs, become more demanding with increasing water and effluent quality goals, it is advantageous to minimize the final flows requiring ultimate treatment.

Thermal discharges. The power industry is an example of an industry that commonly employs recycling of its cooling water via a cooling lake or cooling tower. In the relatively water-abundant C-SELM area, power industry recycling is motivated by a desire to minimize discharge of waste heat to natural waters in order to control the effects of thermal pollution.

There are existing Federal and state standards that specify the conditions under which cooling water discharges are acceptable to receiving waters. Those standards apply not only to the power industry but also to any other potential discharger of waste heat. The C-SELM evaluation of impact of existing and NDCP future standards considers the impact of thermal recycling on the critical industries.

Recycled, blowdown flows from industrial cooling systems are considered acceptable to the C-SELM waterways for the purpose of this study. No deleterious effects are ascribed to the increased total dissolved solids concentrations associated with these blowdown flows.

Solids. The present waste solids management for the major industries is listed in Data Annex B, Section III-A. These generalized management practices are indicative of the present technology being utilized by industries. NDCP water quality standards, which will become effective in the near future, will place greater emphasis on the waste solids management of these industries.

The optimum waste solids management system would involve the reprocessing of the reclaimed solids for use as raw materials or by-products. The ultimate disposal of the non-reclaimable waste solids would utilize a land disposal process which would be compatible with the conservation of the land and water resources.

Reuse potential. The opportunity for the reuse of secondary effluent from municipal treatment plants for industrial process waters may also be developed. The quality of intake waters for a number of C-SELM industries is not significantly different from secondary municipal effluent. At present a number of industries operate raw water pretreatment facilities to properly condition their process water. The incremental cost of pretreating municipal effluent versus present pretreatment operations may prove to be feasible when viewed as a water-conserving technique. Table B-IV-B-1 presents the industrial reuse potential of municipal effluent for a number of industries. There are, however, a number of constraints concerning the practicality of reuse opportunities. For example, the municipal treatment plant should be in close proximity to the particular industry. Industrial reuse of such water may not be feasible in areas where there exists an abundant water supply or where industrial water consumption is much larger than nearby municipal wastewater flows.

INDUSTRIAL CONTRIBUTORS TO MUNICIPAL TREATMENT SYSTEMS

Industrial contributors to municipal treatment systems are projected to increase as NDCP standards are implemented. All present critical and non-critical present dischargers to surface waters are projected to abandon their direct discharges in favor of the more economical transmission of their blowdown flows to municipal treatment systems for final advanced waste treatment. This conclusion is supported by the critical industry analysis described under the next heading of this sub-section.

CRITICAL INDUSTRIES

Steel Industry: Introduction

The water needs of the steel industry per unit of production have increased slightly in recent years reflecting the requirements of new, high-volume production technology. At present and in the foreseeable future, the water requirements of advanced-technology, integrated steel mills will be approximately 40,000 gallons per ton of steel, of which 19,000 gallons per ton, or 47%, is required for indirect cooling; 7,000 gallons per ton, or 18%, is required for direct cooling; and 14,000 gallons per ton, or 35%, is required for process use.

Table B-IV-B-1

INDUSTRIAL REUSE POTENTIAL OF MUNICIPAL EFFLUENT

Industry	Reuse Potential
Steel	For coke and slag quenching, gas cleaning and hot rolling operations, secondary effluent quality would be acceptable. For cold rolling and reduction mill waters, secondary effluent would have to be pre-treated (coagulation, sedimentation, filtration) mainly to reduce suspended solids content. Pickling and cleansing rinse waters require a softened or demineralized water.
Petroleum	Pre-treatment of effluent for suspended solids and turbidity removal is necessary to enable use as process water for desalting, washing and product transportation operations. Utilization of wastewater for brine removal from crude oil produces synergistic effects through wastewater renovation of certain pollutants such as phenols.
Food Processing and Pharmaceutical	The reuse of secondary effluent for process water is not acceptable since all water for washing, transport and blanching operations must be of potable quality.

INDUSTRIAL REUSE POTENTIAL OF MUNICIPAL EFFLUENT

Industry	Reuse Potential
Explosives and Soap	The reuse of secondary effluent for process water would require pre-treatment including coagulation, sedimentation and filtration. Further treatment may include softening and demineralization for the particular desired water quality.
Power and Boiler Feed and Cooling Operations	The reuse of secondary effluent for cooling and boiler feed operations may have limited use. Cooling water use in the steel and petroleum industries far exceeds the process water use in these industries. Pre-treatment will be dependent on specific installations. Generally pre-treatment for boiler feed will be necessary for solids and hardness control.

The distribution of the water needs among the varied sub-processes within advanced-technology steel mills is presented below, together with an indication of the type of pollutant resulting from each of the sub-processes.[1] The waste streams and pollutants from each of the sub-processes have different degrees of recycle potential. The degree of overall recycle possible is evident from the following examples:

(a)   Kaiser Steel Corporation's Fontana, California, integrated plant, where the total water make-up requirement is 1,600 gallons per ton of production.

(b)   Wisconsin Steel Division of International Harvester Company's Cook County, Illinois, integrated plant, where a one-time water intake of 120 MGD has been reduced to 70 MGD and is destined to be reduced to an estimated 5 MGD, or four percent of the original water requirement for the same level of steel production.

(c)   United States Steel Corporation's South Works in Cook County, Illinois, which announced in January, 1971, a recycling program, requiring five years to implement, that will accomplish the reduction of wastewater to a small quantity to be processed by the Metropolitan Sanitary District of Greater Chicago.

A generalized maximum recycle strategy for the integrated steel industry is as follows:

(a)   All cooling flows and the sinter plant, steelmaking processes and hot and cold rolling mill process flows are reclaimed and recycled repeatedly until the total dissolved solids concentration approaches inhibitory levels.

(b)   Blowdown from the recycling flow, described above, is successively used for the by-product coke plant cooling and process requirement followed by the blast furnace process requirement.

(c)   Pickling wastes are regenerated with a hydrochloric acid-thermal-recovery system; tinplating and galvanizing wastes are essentially stripped of their heavy metal contents by adsorption recovery systems and discharged to local or remote and, as required, advanced waste treatment.

(d)   Reclaimed iron solids are recycled to either blast fur-
naces or steelmaking processes via sintering, as required;
reclaimed oil is classified and reused or sold for fur-
ther reclaiming; recovered zinc, tin and chromium are
selectively reclaimed, as economically feasible, and re-
used.

(e)   Sanitary flows are transmitted to local or remote primary,
secondary, and as required, advanced waste treatment.

This example of maximum recycle strategy for the integrated steel
industry leads ultimately to a net water requirement per ton of produc-
tion of approximately 3,000 gallons, or seven and one-half percent of
the present estimated water requirement without recycle. Figure B-IV-B-1
has been prepared to illustrate how such a strategy could develop with
time. The wastewater index presented in this figure is defined as the
fraction of the original wastewater volume per ton of steel production
(40,000 gallons per ton) that accompanied modern technology steel
manufacture prior to wastewater recycle.

A detailed analysis of the steel industry recycle potential follows
under the next heading of this sub-section.

Steel Industry Recycle Potential

Objective and scope . This detailed analysis examines the
status of present wastewater treatment practices within the steel in-
dustry in order that the conditions for compliance with current and
future effluent and water quality standards may be defined. This fur-
ther evaluation of required conditions for compliance demonstrates their
technologic and economic feasibility.

The analysis employs a comparative approach. An integrated
steel mill employing typical production technology with an average pro-
duction capacity is chosen as the modular plant in this study. The
present-day wastewater treatment practices common to the steel industry
are assigned to the modular plant which serves as a base for technologic
improvements and resultant economic variations. With this foundation,
the conditions required by current water quality standards are com-
paratively examined, as are the conditions required to satisfy NDCP
standards.

Present-day processes and treatment technology. At present,
it is estimated that 40,000 gallons of water are required in the produc-
tion of one ton of finished steel in an integrated steel mill. About
53 percent of the water is used in processes and for direct cooling,
while the remaining 47 percent is used for indirect cooling. The mod-
ular steel mill employs typical present steel technology [1] with a
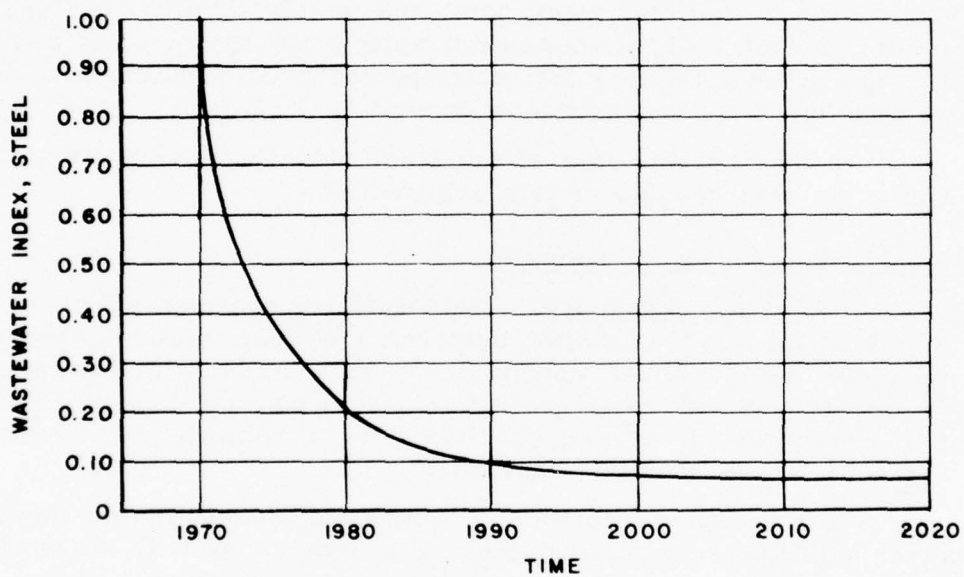
Figure B-Ⅳ-B-1

THE EFFECT OF WASTEWATER RECYCLING IN THE
STEEL INDUSTRY ON TREATMENT VOLUME REQUIREMENTS

B-Ⅳ-B-8

production capacity of 1.5 million tons of finished steel per year (4,110 tons/day). The block diagram in Figure B-IV-B-2 shows present process and treatment technology. Table B-IV-B-2 lists the flows and waste loads common to each sub-process presented in Figure B-IV-B-2. The present treatment technology reflects selected practices now occurring in the C-SELM area superimposed upon generalized nation-wide treatment technology.

A review of steel industries in the C-SELM area shows that the majority of steel mills are located in Northwestern Indiana. The intake water for a typical mill is taken from Lake Michigan and discharges eventually enter Lake Michigan by way of improved channels entering the harbor area described by Indiana Water Quality Standards as the inner Harbor.[2] These standards are adopted to represent the current standards in this study, and those that specifically apply to steel mill wastewaters are listed in Table B-IV-B-3.

Present treatment of process and direct cooling flows at the modular plant produces effluent waste streams with the characteristics shown in Table B-IV-B-4. Efficiencies for the processes were taken from the Cost of Clean Water, Steel Industry.[1] At present there are no provisions for removal of heat from hot process or cooling waters. The heat load to water is estimated to be about 5 million BTU/ton of finished steel.[3] For a total water use of 164 MGD in the modular plant, the average increase in temperature of the effluent is $15°F$. On the basis of steel produced in the C-SELM area, about 20.5 billion BTU are dissipated in effluent each day.

A comparison of present effluent wastewater quality with current water quality standards must consider dilution by the receiving water. Most discharges spend only a short time in-stream before entering the Inner Harbor. Also, the channels accepting these discharges carry flow composed almost entirely of discharges from the dense industrial sector in near proximity of the Inner Harbor. Due to these conditions, the quality of the flows in these accepting channels limits the addition of further increments of pollutants to relatively small amounts. Furthermore, the effects of mixing are limited once the stream enters the lake. A plume diffusion analysis, employing dimensionless quantities for both channel and lake dimensions, reveals that for the estimated flows in the area, the dispersion of suspended, soluble, and heat quantities in the stream is only about twenty percent of the total along a radius of 1,000 feet. Once-through use and discharge, as is now practiced, places severe limitations as to the dilution expected by discharging into the lake. Dilution advantages for specific steel mill discharges, are therefore, assigned on a judgment basis with consideration given to upstream conditions and current water quality standards. An example of the plume diffusion analysis is presented in Data Annex B, Section IV-B.

| Blast Furnace & Sinter Plant (Iron Production) | Oxygen Lanced Open Hearth Furnace (Steel Production) | Hot Rolling Mills | Cold Rolling Mills | Steel Fin (Pickling & |

Flue Gas Wash Water — 10 MGD
Furnace Cooling Water — 30.8 MGD
Flue Gas Wash Water — 6 MGD
Furnace Cooling Water — 22.5 MGD
Scale Removing Cooling Water — 30 MGD
Furnace & Equipment Cooling Water — 14.5 MGD
Flood Lubricating Emulsions — 12.3 MGD
Spent Pickle Liquor — 35,000 GPD
Pickle Liquor Rinsewater — 90,0 GP

PRESENT DAY TREATMENT OF WASTEWATERS FROM AN INTEGRATE

125,000 GPD

16 MGD

30 MGD

| Clarifier (For Flue Dust Wash Water From Blast Furnace, Sinter Plant, and Oxygen Lanced Furnace) |

16 MGD Discharge to Receiving Waters

| Scale Pit Clarifier (For Hot Rolling Mill *Scale*) |

30 MGD Discharge to Receiving Waters

| Pickle Liquor Neutralization (HCl) (Spent Liquor & Rins |

Lagoon Clarification

Sludge Disposal (by Land Use)

Underflow Suspended Solids to Dewatering (Vacuum Filter or other) & Sinter Plant

78.2 MGD

Once Through Indirect Cooling Waters

78.2 MGD Discharge to Receiving Waters

12.3 MGD

| Coagulation and Flotation of Cold Mill Emulsions |

12.3 MGD Discharge to Receiving Waters

Sludge Disposal (by Land Use)

Total Intake = 164.3 MGD = Total Discharge

| g Mills | Cold Rolling Mills | Steel Finishing (Pickling & Plating) | Sanitary Wastes | By-Product Coke Plant |

Furnace & Equipment Cooling Water

Flood Lubricating Emulsions

Spent Pickle Liquor

Pickle Liquor Rinsewater

Chrome Rinsewater

In Plant Sanitary Use

Process & Quench Water

Equipment Cooling Water

14.5 MGD 12.3 MGD 35,000 GPD 90,000 GPD 72,000 GPD 7.6 MGD 20 MGD 10.4 MGD

EATMENT OF WASTEWATERS FROM AN INTEGRATED STEEL MILL

125,000 GPD

72,000 GPD

30 MGD Discharge to Receiving Waters

Pickle Liquor Neutralization (HCl) (Spent Liquor & Rinse)

Lagoon Clarification

Sludge Disposal (by Land Use)

125,000 GPD Discharge to Receiving Waters

Ion Exchange for Chromium

72,000 GPD Discharge to Receiving Waters

Regeneration Wastes

12.3 MGD

27.6 MGD

Coagulation and Flotation of Cold Mill Emulsions

Sludge Disposal (by Land Use)

12.3 MGD Discharge to Receiving Waters

Biological Treatment in Aerated Lagoons (Coke Plant & Sanitary Wastes)

Sludge Disposal (by Land Use)

27.6 MGD Discharge to Receiving Waters

Figure B-IV-B-2

PRESENT STEEL MILL
WASTEWATER PRACTICE

B-IV-B-10

Table B-IV-B-2

1.5 MILLION TONS ANNUAL CAPACITY TYPICAL STEEL MILL – WATER USE AND POLLUTANT LOADINGS

Sub-Process	Flows (MGD) Process & Direct Cooling	Indirect Cooling	Pollutant Loadings, Pounds Suspended Solids	Oils	H_2SO_4	$FeSO_4$	Cr^{+6}	NH_3	CN	Phenols	Heat $\times 10^6$ BTU/Ton
Blast Furnace & Sinter Plant	10	30.8	283,000								1.135
Oxygen Lanced Open Hearth Furnace	6	22.5	115,000								.760
Hot Rolling Mills	30	14.5	230,000	11,200							1.785
Cold Rolling Mills	12.3	–	570	1,700							.248
Spent Pickle Liquor	0.04				9,200	38,500					
Pickle Liquor Rinse	0.09	–			1,620	6,800					
Chrome Rinse	0.07	–					259				
By-Product Coke Plant	20	10.4		600				3,640	370	2,620	1.025
Sanitary & Other	7.6		$BOD_5 = 200$ mg/l								
TOTALS	86.1	78.2	628,570	13,500	10,820	45,300	259	3,640	370	2,620	5×10^6

B-IV-B-11

Table B-IV-B-3

CURRENT WATER QUALITY STANDARDS -
INDIANA - INNER HARBOR REGULATIONS

(Parameters Relating to Steel Mill Discharges Only)

mg/l

Ammonia	Chlorides	Cyanides	Iron	Oils	Phenols	Phosphates	Chrome	Total Dissolved Solids	Suspended Solids	Temp.[a]
0.05	20	0.025	0.15	None visible	0.002	0.03	0.05	197	No increase above background concentration	
								167[c]	8[c]	

Month	Max. Temp-°F	Ambient-°F[b]
January	45	40
February	45	35
March	45	40
April	55	45
May	60	52
June	70	58
July	80	63
August	80	68
September	80	66
October	65	63
November	60	55
December	50	46

[a]At any time and at a maximum distance of 1,000 feet from a fixed point adjacent to the discharge and or as agreed upon by the State and Federal regulatory agencies, the receiving water temperature shall not be more than 3 degrees Fahrenheit above the existing natural water temperature nor shall the maximum temperature exceed those listed below whichever is lower.

[b]Assumed Ambient Lake Michigan Temperatures.

[c]Reported Lake Michigan Intake Water Quality.

Table B-IV-B-4

1.5 MILLION TONS ANNUAL CAPACITY TYPICAL STEEL MILL
EFFLUENT QUALITY OF SPECIFIC PROCESS AND DIRECT COOLING
WASTE STREAMS AFTER PRESENT TREATMENT
(Loadings in Pounds (#), Concentrations in mg/l)

Stream	Flow, MGD	Suspended Solids	Oils	Chlorides	Iron	Cr^{+6}	NH_3	CN^-	Phenols[a]
Blast Furnace & Sinter Plant	10	17,600# 211 mg/l	-						
Oxygen Lanced Open Hearth	6	7,100# 142 mg/l	-						
Hot Rolling Mills	30	23,000# 92 mg/l	8,960# 36 mg/l						
Cold Rolling Mill	12.3	26# 0.25 mg/l	85# 0.8 mg/l						
Spent Pickle Liquor	0.04			6,090# 20,900mg/l	3,400# 11,700mg/l				
Pickle Liquor Rinse	0.09			5,370# 7,150mgl	3,000# 4,000 mg/l				
Chrome Rinse	0.07					13# 22 mg/l			
By-product Coke, Sanitary & Other	27.6		600# 2.6mg/				728# 3.1 mg/l	74# 0.3 mg/l	524# mg/l
TOTALS	86.1	47,700#	9,645#	11,460#	6,400#	13#	728#	74#	524#
Concentration in mg/l if Total Plant Flow Used for Dilution (164 MGD)[b]		35	7	8	5	0.01	0.5	0.05	0.4

[a] BOD5 reduced from 200 mg/l to 40 mg/l.

[b] Design of additional treatment facilities was based on specific stream concentrations, with dilution benefits used only if streams are combined before discharge.

Treatment required to satisfy current water quality standards.
A comparison of Tables B-IV-B-3 and B-4 indicates the problem areas in
present treatment. The intake water quality, as reported by the indi-
vidual C-SELM mills, provides the required background concentrations
in Lake Michigan water for application of specific standard requirements.

Suspended solids in clarifier effluents are much higher than back-
ground concentrations. Under current standards these concentrations
must be reduced to 8 mg/l. Since waters carrying suspended solids
also carry a considerable amount of heat, it is necessary to also cool
these waters before discharge. Conventional clarification followed
by high rate sand filters[4] and cooling towers[5] is chosen as the treat-
ment scheme required to satisfy current standards. Current standards
allow blowdowns from closed type cooling systems to exceed the im-
posed temperature limitations. Therefore, a recirculation scheme was
added to the above treatment. Recirculation was limited by the dissolved
solids buildup due to additions in process and from tower evaporation.
The recirculation scheme minimizes the size of the additional treatment
facilities due to the reduced hydraulic loadings. It is assumed that hot
rolling processes may demand stringent temperature requirements, and
these requirements are assumed to be met by the flexibility of the re-
circulation scheme. Cooling tower blowdown is used for coke quenching
operations.

Figure B-IV-B-3 shows the anticipated treatment and recirculation
scheme for these streams and serves as the basis upon which sizing
and costing are determined. Indirect cooling water recirculation and
cooling schemes are also shown in Figure B-IV-B-3, with recirculation
limited by a maximum practical dissolved solids concentration.

From the preliminary analysis, it is felt that the ammonia con-
centration now present in effluent discharges satisfies current standards.
Iron and chloride concentrations are very high in pickling operation
discharges. Spent hydrochloric acid pickle liquor is regenerated in a
thermal regeneration process now commonly marketed, with cost bene-
fits accruing from the recovery of hydrochloric acid.[6] Pickling rinse
water is neutralized with lime and added to the coke plant and sani-
tary waste streams.

Phosphorus removals of 80 percent are required by the State of
Indiana for discharges to Lake Michigan. The treatment required for
coke plant, sanitary, and pickling rinse wastes includes BOD and sus-
pended solids reduction, phosphorus removal, phenol reduction and iron
removal. Reduction of BOD, suspended solids, cyanides, and ammonia

FURNACE AND CONDENSER COOLING WATERS

Diagram text elements:

283,000 #/day SS
2.1x10^9 BTU/Day
200 mg/l TDS

4.6 MGD Blowdown
to Coke Quenching Operations

1.38 MGD Evaporation

Make Up Water
1.3 MGD
170 mg/l TDS
68°F.
8 mg/l SS

Blast Furnace — 10 MCD 136°F — Clarifier 93% Efficient — Deep Bed Filters (Pressurized)
5 mg/l SS
5 ppm Oil
3.5 MGD
136°F.

5.5 MGD–136°F.

3.2 MGD–86°F.

Cooling Tower
Assumed Evaporation is 3% of total Inflow.
Exit Water Temperature is 86°F.

5 mg/l SS
5 ppm Oil

5 mg/l SS
5 ppm Oil

Deep Bed Filters (Pressurized)

11.9 MGD
86°F.

Clarifier
Scale Pit

14.2 MGD
110°F.

30 MGD

Make Up Water
3.9 MGD
170 mg/l TDS
68°F.
8 mg/l SS

Hot Rolling Mill

230,000 #/Day SS
3.75x10^9 BTU/Day
200 mg/l TDS

FURNACE AND CONDENSER COOLING WATERS

63.7 MGD
@ 126°F.
2000 mg/l TDS

5.3x10^9 BTU/Day

Makeup Water
0.414 MGD
68°F.
170 mg/l TDS

Furnace & Condenser — 63.7 MGD 136°F. 2000 mg/l TDS — Cooling Tower

Blowdown
0.034 MGD
2063 mg/l TDS
86°F.

51.1 MGD–136°F.–2000 mg/l TDS

12.2 MGD–86°F.–2062 mg/l TDS

Evaporation
0.38 MGD

NOTES

1. A maximum temperature limit of 136°F. is placed on all heat bearing waters.
2. In the case of hot rolling mills, where the temperature is considered to be a controlling factor in the process, the maximum allowable temperature of 110°F. is assumed.
3. Process and direct cooling waters must be cooled on a separate tower from the indirect cooling waters if maintenance problems in the cooling system are to be kept at a minimum.
4. A variation in the temperature restraints of any flow mill will shift the quantity of flow in any one recirculatory cycle to a new steady state condition.
5. An evaporation rate of 3 percent of the flow to the cooling tower is assumed.
6. High rate, deep bed filters are used under pressure in order to avoid subsequent pumping to the cooling tower.
7. All temperatures are assumed as average maximum summer temperatures.
8. TDS = Total Dissolved Solids, SS = Suspended Solids

SUSPENDED SOLIDS AND HEAT TREATMENT

4.6 MGD
Blowdown

1.38 MGD
Evaporation

115,000 #/Day SS

1.24×10^9 BTU/Day
200 mg/l TDS

Coke Quenching
erations

5 mg/l SS
5 ppm Oil
3.5 MGD
136°F.

Cooling Tower

Assumed Evap-
oration is 3% of
total Inflow.

Exit Water Temp-
erature is 86°F.

5 mg/l SS
5 ppm Oil

Deep Bed
Filters
(Pressurized)

Clarifier
93%
Efficient

6 MGD
136°F.

Steel
Furnace

Make Up
Water
0.78 MGD
170 mg/l TDS
68°F.
8 mg/l SS

3.3 MGD-136°F.

1.92 MGD-86°F.

5 mg/l SS
5 ppm Oil

11.9 MGD
86° F.

Deep Bed
Filters
(Pressurized)

Clarifier
Scale Pit

14.2 MGD
110°F.

30 MGD

Make Up Water
3.9 MGD
170 mg/l TDS
68°F.
8 mg/l SS

Hot
Rolling
Mill

230,000 #/Day SS
3.75×10^9 BTU/Day
200 mg/l TDS

PRE-HEAT AND RE-HEAT FURNACE COOLING WATERS
(HOT ROLLING MILLS)

3.63×10^9 BTU/Day

14.5 MGD
@ 106°F.
2000 mg/l TDS

Make Up
Water
0.283 MGD
68°F.
170 mg/l TDS

Pre-Heat
&
Re-Heat
Furnace

14.5 MGD
136°F.
2000 mg/l TDS

Cooling
Tower

Blowdown
0.023 MGD
2026 mg/l TDS
86°F.

5.90 MGD-136°F.-2000 mg/l TDS

8.32 MGD-86°F.-2062 mg/l TDS

Evaporation

0.26 MGD

earing waters.
nsidered to be a controlling
10°F. is assumed.
e tower from the indirect
are to be kept at a minimum.
ll shift the quantity of
dition.
ower is assumed.
to avoid subsequent pumping

atures.

Figure B-Ⅳ-B-3

RECIRCULATION DESIGN FOR
STEEL MILL COOLING FLOWS

are adequate under present practice. Phenols are not reduced to satisfactory levels and must be further reduced by carbon adsorption. Phosphorus is reduced by lime precipitation followed by clarification. The iron concentration is reduced in the same manner, although not as completely. The addition of neutral pickling rinsewater to the coke and sanitary streams somewhat reduces later lime requirements for both phosphorus and iron removal. In order to maximize reuse, an additional 15.4 mgd of effluent from the biological treatment, aerated lagoon system is recycled to the by-product coke plant to satisfy the remaining process and quench needs of this part of the steel mill.

The alternatives for achieving the treatment requirements given above are on-site treatment utilizing existing facilities or piping the wastewater to a regional advanced wastewater treatment plant. The advantage for on-site treatment is the presence of an existing biological treatment facility. The disadvantages include the extra land requirements for additional processes. The advantage in piping the waste to a regional facility is due mainly to the benefit of economy, of scale associated with the various regional treatment process components. Disadvantages include the extra cost of piping the wastes from the mill to the treatment plant and the necessity to treat the flow in all of the regional facility advanced processes since selective separation of flows is not possible. On-site treatment is found to be less expensive than treatment at a regional facility for the achievment of current effluent standards. The on-site treatment includes existing aerated lagoons, lime precipitation of phosphorus and iron and carbon adsorption of regional organics including phenols. The effluent stream from these processes must be aerated prior to its confluence with the receiving stream. Post-aeration has, therefore, been included.

Chrome wastes cannot be reduced beyond the efficiency of the existing ion exchange process unless deep well disposal is practiced. The latter, however, is not considered to be a universal, long-range abatement practice.

Cold mill emulsions are treated for palm oil recovery by a series of physical and chemical operations. The underflow from this operation is recycled for reuse in the cold rolling process. Blowdowns, when required, are added to the biological treatment facilities. The treatment facilities, listed here to meet current standards, are shown in Figure B-IV-B-4.

Due to recirculation and reuse of certain effluent streams, the total water intake with these treatment facilities is decreased to 29.9 MGD and discharges amount to 27.8 MGD. The 2 MGD difference is due to evaporation losses. Effluent quality for wastewaters treated by these facilities to meet current standards is shown in Table B-IV-B-5. Comparing Tables B-IV-B-4 and B-IV-B-5 demonstrates the additional removal of certain pollutants by intensifying the degree of treatment beyond present practice.

B-IV-B-16

TYPICAL PRESENT-DAY TECHNOLOGY FOR AN INTEGRATED
STEEL MILL OF 1.5 MILLION TONS ANNUAL CAPACITY

≡PROCESSES, WASTE STREAMS, AND VOLUMES≡

| Blast Furnace & Sinter Plant (Iron Production) | Oxygen Lanced Open Hearth Furnace (Steel Production) | Hot Rolling Mills | Cold Rolling Mills | Steel Finish (Pickling & Pl |

Flue Gas Wash Water — 10 MGD

Furnace Cooling Water — 30.8 MGD

Flue Gas Wash Water — 6 MGD

Furnace Cooling Water — 22.5 MGD

Scale Removing Cooling Water — 30 MGD

Furnace & Equipment Cooling Water — 14.5 MGD

Flood Lubricating Emulsions — 12.3 MGD

Spent Pickle Liquor — 35,000 GPD

Pickle Liquor Rinsewater — 90,000 GPD

TREATMENT REQUIRED TO MEET PRESENT STANDARDS

16 MGD

30 MGD

78.2 MGD

57 MGD
Hot Wa
Recycl
Coolin

21.2 MGD

20.
Coo
Rec
Coo

CLARIFIER (For Flue Dust Wash Water From Blast Furnace, Sinter Plant, and Oxygen Lanced Furnace)

Clarifier Overflow Recycled to Flue Gas Washers 23 MGD

Scale Pit

CLARIFIER (For Hot Rolling Mill Scale)

Underflow Suspended Solids to Dewatering (Vacuum Filter) and Sinter Plant

Cooling Tower (For Indirect cooling water)

Blowd
0.06
Disch
Receiv

0.64 MGD Evaporation

Underflow Suspended Solids to Dewatering (Vacuum Filter) and Sinter Plant.

23 MGD

Filter Backwash Plus Polymer to Clarifier

High Rate Filter (Under Pressure for Removal of Suspended Solids and Oils)

Cooling Tower (For Indirect Cooling Water)

17 MGD Cooled Water Recycled to Flue Gas Washers

12.3 MGD

12.3 MGD Recirculated to Cold Rolling Mill

Palm Oil Recovery Units (From Cold Rolling Mill)

Blowdown 4.6 MGD Reuse for By-Product Coke Plant Process Water or Quench Water

1.38 MGD Evaporation

Salvaged Oil for Reuse

Sludge Disposed at Regional AWT Plant

Total Intake = 29.9 MGD
Total Discharge = 12.4 MGD

lls	Cold Rolling Mills	Steel Finishing (Pickling & Plating	Sanitary Wastes	By-Product Coke Plant

Cooling Water

Flood Lubricating Emulsions

Spent Pickle Liquor

Pickle Liquor Rinsewater

Chrome Rinsewater

In Plant Sanitary Use

Process & Quench Water

Equipment Cooling Water

MGD   12.3 MGD   35,000 GPD 90,000 GPD 72,000 GPD   7.6 MGD   20 MGD   10.4 MGD

T REQUIRED TO MEET PRESENT STANDARDS

90,000 GPD   7.6 MGD   20 MGD

78.2 MGD   ↑ 57 MGD Hot Water Recycled to Cooling Coils

21.2 MGD   ↓ 20.5 MGD Cool Water Recycled to Cooling Coils

Underflow Suspended Solids to Dewatering (Vacuum Filter) and Sinter Plant

Lime Neutralization

15.4 MGD Recycled To By-Product Coke Plant

Sludge

Cooling Tower (For Indirect cooling water)

Blowdown 0.06 MGD Discharge to Receiving Waters

On Site Treatment Aerated Lagoons Plus 80% Phosphorus Removal Plus Carbon Column Polishing

12.3MGD Discharge to Receiving Waters

Post Aeration

0.64 MGD Evaporation

Sludge Disposed at Regional AWT Plant

17 MGD Cooled Water Recycled to Flue Gas Washers

12.3 MGD

12.3 MGD Recirculated to Cold Rolling Mill

35,000 GPD

Regenerated HCl

72,000 GPD

lowdown .6 MGD

Palm Oil Recovery Units (From Cold Rolling Mill)

Spent Pickle Liquor, Thermal Regeneration Unit (Hydro-chloric Acid)

Ion Exchange For Chromium

72,000 GPD Discharge to Receiving Waters

use for Product Coke Process Water ench Water

Regeneration Wastes Stored for Possible Future Chrome Recovery

Salvaged Oil for Reuse

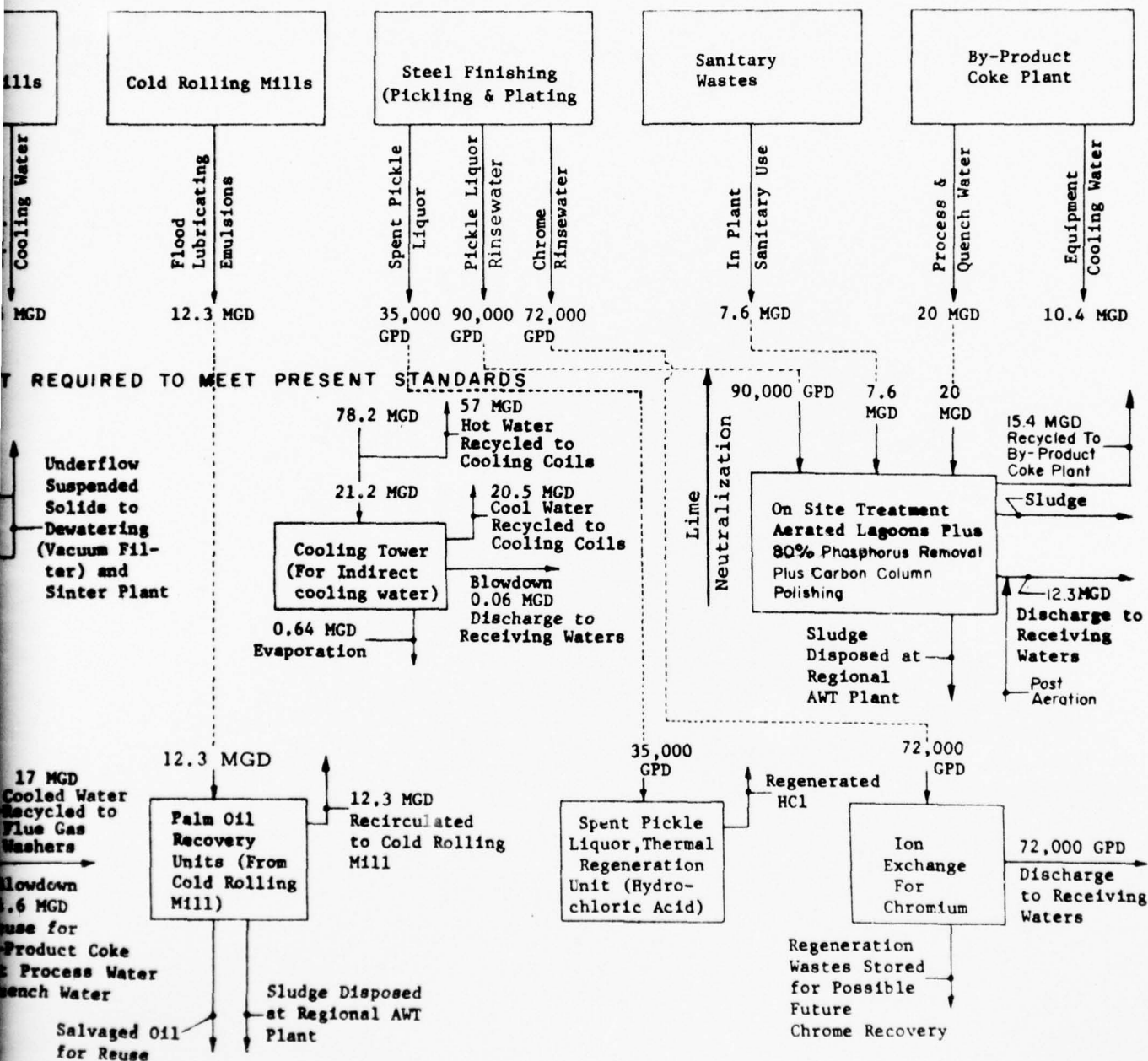Sludge Disposed at Regional AWT Plant

Figure B-ⅣV-B-4

STEEL MILL WASTEWATER
PRACTICE WITH CURRENT STANDARDS

Table B-IV-B-5

1.5 MILLION TONS ANNUAL CAPACITY TYPICAL STEEL MILL - EFFLUENT QUALITY AFTER TREATMENT TO MEET CURRENT STANDARDS
(Loadings in Pounds (#), Concentrations in mg/l)

Stream	Flow, MGD	Suspended Solids	Oils	Chlorides	Iron	Cr^{+6}	NH_3	CN^-	Phenols[a]
Blast Furnace & Sinter Plant	1	42# 5 mg/l	42# 5 mg/l						
Oxygen Lanced Open Hearth	0.6	25# 5 mg/l	25# 5 mg/l						
Hot Rolling Mills	3	125# 5 mg/l	125# 5 mg/l						
Cold Rolling Mill	0								
Spent Pickle Liquor	0								
Chrome Rinse	0.07					13# 22 mg/l			
By-product Coke Plant, Sanitary Wastes, & Pickle Liquor	27.7		230# 1 mg/l	5,370# 22 mg/l	3,000# 13 mg/l		728# 3.1 mg/l	0	2.3# 0.01 mg/l
Rinse	0							0	
TOTALS		192#	422#	5,370#	3,000#	13#	728#	0	2.3#

[a] BOD_5 is reduced from 200 mg/l to 3 mg/l

Although the temperature of the discharged cooling waters is higher than it would be without treatment, the volume is reduced from 124.2 MGD to 4.66 MGD, of which 4.6 MGD is further reused for coke quenching water. Assuming a final discharge of 4.66 MGD at the increased temperatures, the heat removed is found to be 15.95 billion BTU/day, compared to a daily input of 20.5 billion BTU. This is a reduction of 78% of the total heat load accomplished by treating 76% of the initial intake volume.

Treatment required to meet NDCP. The most complete application of presently-available conventional and advanced treatment technology is capable of producing effluents of a quality listed in Table B-IV-B-6. A comparison between current standards and NDCP presents a problem because of the dilution opportunities associated with current standards. NDCP standards are compared directly to the effluent quality of wastewaters treated under the system to satisfy current standards. A comparison of Table B-IV-B-5 and B-IV-B-6 indicates that ammonia and solids removals must be increased to meet NDCP requirements. Suspended solids, in blowdown streams from the cooling towers, are slightly higher in concentration than that which is required by NDCP, but it is not practical to treat for solids alone. Also, the background suspended solids concentration in untreated intake water is slightly higher than in the effluent. Blowdowns from cooling towers are high in total dissolved solids due to recirculation and evaporation. Reverse osmosis or ion exchange technology is available for total dissolved solids separation, but it is expensive and does not provide for the ultimate disposal of the separated solids. Another point of view emphasizes that the quantity of total dissolved solids remains the same as at the point of intake, because the total quantity of water is reduced through evaporation. The plume diffusion analysis referred to earlier indicates that the higher total dissolved solids concentration will not dissipate rapidly due to limited turbulence for mixing. The localized effect, however, of a larger quantity of dissolved solids is not, as far as is known, deleterious to the desired receiving water.

Iron removal depends on pH and other conditions of ionic equilibria. Since iron removal is a by-product of phosphorus removal treatment, the iron remaining in the effluent after NDCP treatment for phosphorus removal will be less than that remaining in the current standard effluent after phosphorus removal.

As in treatment to meet current standards, in order to maximize reuse, an additional 15.4 MGD of effluent from the biological treatment, aerated lagoon system is recycled to the by-product coke plant to satisfy the remaining process and quench needs

Table B-IV-B-6

NO DISCHARGE OF CRITICAL POLLUTANT PERFORMANCE CRITERIA

Effluent Quality

Treatment Type	COD mg/l	BOD mg/l	Suspended Solids mg/l	Soluble Phosphorus mg/l	NH3-N mg/l	NO3-N NO2 mg/l	Organic N mg/l	Heat, Temp. °F	Oils Greases mg/l	Phenols mg/l	Pathogens, Viruses	Accum. Trace Metals[a] mg/l	Boron mg/l	Arsenic mg/l	Cyanide mg/l
Advanced Biological	10	3	1	0.1-0.2	0.3	2-5	b	53-78	1	0.01	Present[b]	0.1	1.0	0.03	0
Chemical-Physical	10	3	1	0.1-0.2	0.5	2	0	53-78	1	0.01	Present[b]	0.1	1.0	0.03	0
Land Treatment	6	2	0	0.01	0	2	0	55-70	0	0	0	0	0	0	0

[a] Trace Metals: Aluminum, Cadmium, Chromium, Copper, Lead, Nickle, Zinc, Iron, Manganese, Mercury.

[b] Present with Current Disinfection Practice.

of this part of the steel mill.

The above comparison indicates that all that is required to pro-
gress from current standards treatment to NDCP treatment is the com-
pletion of the AWT portion that treats the net 12.3 MGD blowdown from
the coke, sanitary, and pickling wastes and the 0.07 MGD of chrome
recovery wastes. This includes either biological or ion exchange am-
monia removal, filtration, and an increase in phosphorus removal from
80% to 98%. Total reduction in unit flow is 92.5%; therefore, 7.5%
of the original unit flow of wastewater per unit of steel production
requires treatment. This agrees with the unit wastewater reduction
data provided in Appendix B, Section III-B.

Recovery of usable materials. During the treatment of certain
waste streams by a specific treatment process, the recovery of usable
materials from that waste stream is made possible. Sedimentation
produces a large quantity of iron oxide which can be sintered and re-
made into steel.

Palm oil recovery units recover the oils for flood lubricating
in cold rolling operations. Thermal regeneration of pickle liquor
waste streams generally recovers a great volume of the spent hy-
drochloric acid used for finishing the steel.

All of the materials recovered from the waste streams pro-
vide an economic benefit to the industry which can be credited to
the economy of the related wastewater treatment technology. This
is discussed further in Appendix B, Section VI-B.

Petroleum Industry: Introduction

In the past fifteen years, the potential make-up water require-
ment for crude oil for both processing and cooling has decreased from
440 to 60 gallons per barrel. This has been possible largely through
recycle of cooling water.[7,8] Further reduction in wastewater production
per barrel of crude is still possible.

The waste characteristics of the various petroleum production
sub-processes are illustrated in Table B-IV-B-7. Each of the waste-
water parameters is compatible with conventional primary, secondary
and advanced waste treatment, as required. Pretreatment for oils and
sulfides is frequently required.

A review of large (greater than 5 MGD effluent discharges)
petroleum refineries in the C-SELM study area reveals that recycle of
cooling water is not intensively practiced and that the potential reduc-
tions referred to earlier have not been achieved. With an ultimate

WASTEWATER CHARACTERISTICS IN THE VARIOUS
SUB-PROCESSES OF THE PETROLEUM INDUSTRY

Fundamental Processes	Flow	BOD	COD	Phenols	Sulfide Oil	Emulsified Oil	pH	Temp.	Ammonia	Chlorides	Acidity	Alkalinity	Susp. Solids
Crude Oil and Product Storage	X	X	X		X	X							X
Crude Oil Desalting	X	X	X	X	X	X	X	X	X	X		X	X
Crude Oil Distillation	X	X	X	X	X	X	X	X	X	X		X	X
Thermal Cracking	X	X	X	X	X		X	X	X	X		X	X
Catalytic Cracking	X	X	X	X	X	X	X	X	X	X		X	X
Hydrocracking	X				X			X					
Reforming	X			X	X			X	X				
Polymerization	X	X	X		X		X	X	X	X	X		X
Alkylation	X	X	X		X		X	X	X	X	X		X
Isomerization	X												
Solvent Refining	X	X	X	X		X	X					X	
Dewaxing	X	X	X	X	X								
Hydrotreating	X	X	X		X		X					X	
Drying and Sweetening	X	X	X	X	X		X		X		X	X	X

X -indicates presence of flow constituents

Blank indicates absence of constituent
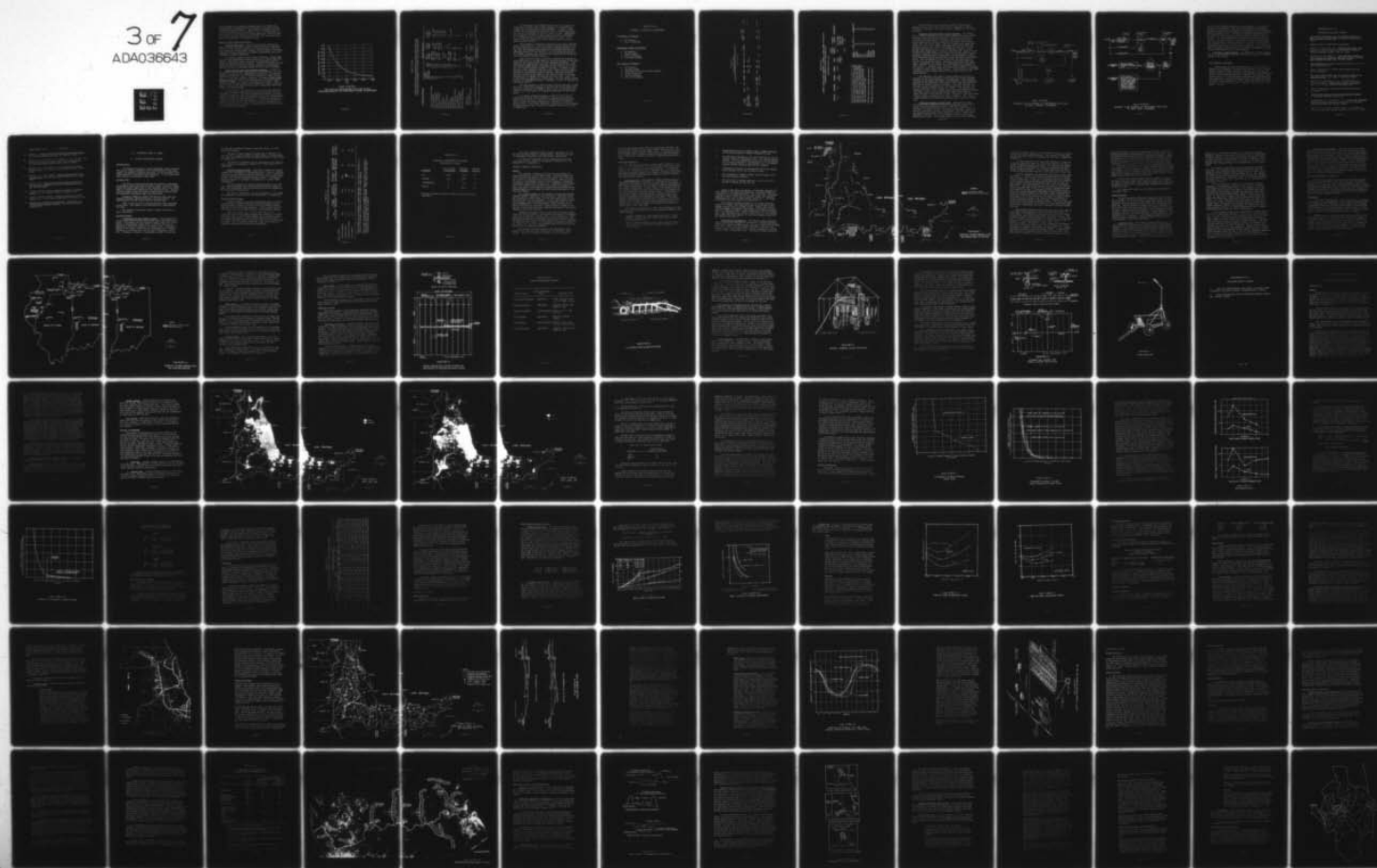
recycle strategy in the petroleum industry within the C-SELM area, it is possible to hypothesize a major reduction in petroleum industry wastewater requiring treatment. Figure B-IV-B-5 illustrates how such a recycle strategy could be achieved over time. The wastewater index is defined as the fraction of original wastewater volume per barrel of petroleum production (440 gallons per barrel) that accompanied petroleum processing prior to wastewater recycle.

Petroleum Industry Recycle Potential

Objective and scope. This detailed analysis examines the status of present wastewater treatment practices within the petroleum industry in order that the conditions for compliance with current and future effluent and water quality standards may be defined. The further evaluation of these conditions for compliance demonstrates their technological and economic feasibility.

The analysis assigns typical process technology, average capacity and current wastewater treatment practices to a modular petroleum refinery. This module serves as a base for technological improvements and resultant economic variations. With this foundation, the conditions required by current water quality standards are comparatively examined, as are the conditions required to satisfy NDCP standards.

Present-day processes and treatment technology. A review of the literature indicates that quantification of such parameters as plant capacity, sub-process mix, wastewater characteristics, and treatment facilities is difficult in view of the great diversity in design and purpose of the individual petroleum refineries. The Cost of Clean Water, Petroleum Industry [7] provides the characteristics of a typical present day refinery based on a review of the nation's oil industry. Table B-IV-B-8 lists the most probable sub-processes for a plant of 100,000 barrels per stream day (bpsd) capacity, with corresponding wastewater volumes and waste loadings.

On the basis of this information it can be projected that about 100 gallons of process wastewater are produced during the refining of a barrel of crude oil. From values reported by the industry in recent years, the indirect and direct cooling water requirement is approximately four times the process wastewater flow[8] for a total use requirement of about 44 MGD in a 100,000 bpsd plant. Wastewater discharges reported by specific C-SELM area refineries indicate a significant lack of recirculation of cooling waters in comparison with the national cross-section of petroleum refineries.
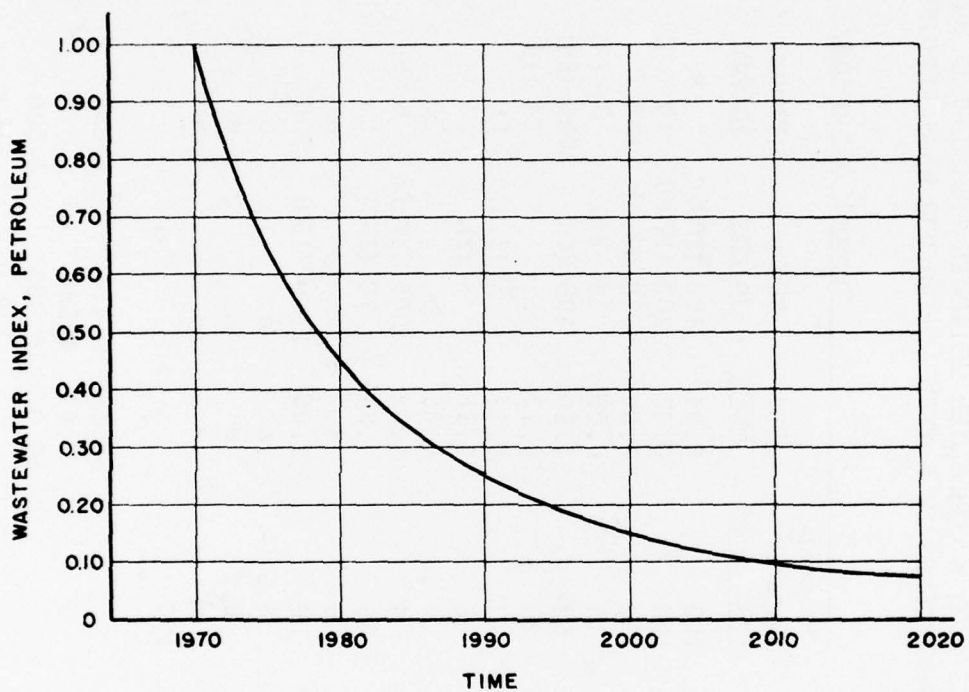
Figure B-IV-B-5

THE EFFECT OF WASTEWATER RECYCLING IN THE
PETROLEUM INDUSTRY ON TREATMENT VOLUME REQUIREMENT

B-IV-B-24

Table B-IV-B-8

WASTE LOADINGS AND WASTEWATER VOLUMES ASSOCIATED WITH
FUNDAMENTAL PROCESSES IN A TYPICAL 100,000 bpsd REFINERY

Fundamental Process	Process Capacity 1000 bpsd	Flow MGD	Typical Technology		
			BOD5 lb/day	Phenol lb/day	Sulfides lb/day
Crude Oil Product Storage	100	0.04	100 (300)	a	a
Crude Desalting	100	0.02	200 (1200)	10 (60)	200 (1200)
Crude Fractionation	100	5.00	20 (0.4)	100 (2)	100 (2)
Thermal Cracking	15	0.03	15 (60)	3 (12)	15 (60)
Catalytic Cracking	50	1.50	500 (40)	1000 (80)	150 (12)
Reforming	20	0.12	t	14 (14)	20 (20)
Polymerization	1	0.14	3 (2.5)	t	10 (8)
Alkylation	6	0.36	6 (2)	1	60 (20)
Solvent Refining	6	0.05	a	18	t
Dewaxing	4	0.09	2000 (2700)	6 (9)	t
Hydrotreating	35	0.04	70 (280)	t	70 (280)
Deasphalting	3	a	a	a	a
Drying and Sweetening	50	2.00	2500 (150)	500	a
Wax Finishing	0.5	a	a	a	a
Grease Manufacture	0.2	a	a	a	a
Lube Oil Finishing	4	a	a	a	a
Blending and Packaging	100				
Summation		9.39	5,414	1,652	625
Unaccounted		.61	4,586		
Total Plant Effluent		10.0	10,000	2,000 b	1,000 b
			120 c	24 c	12 c

a - data not available for reasonable estimate   b - approximation due to unknown conditions
t - trace   ( )- concentrations in mg/l
c - extreme loading estimate

The locations of the C-SELM refineries fall mostly within the jurisdiction of Indiana Water Quality Standards for wastewater discharges into the inner harbor of Lake Michigan. A typical C-SELM refinery has been selected on that basis, with a 100,000 bpsd capacity and the sub-process mix listed in Table B-IV-B-8. Reassignment of the additional 34 MGD of intake water among the sub-processes for a C-SELM refinery is beyond the scope of this study. It is assumed that BOD, phenol and sulfide loadings remain as they are shown in Table B-IV-B-8.

Three degrees of treatment are provided at the typical refineries listed in Table B-IV-B-9. The high degree of treatment appears to be the most representative because public and government pressure in recent years has prompted many industries to accelerate their pollution abatement practices in this direction. Also, the discharge records of the C-SELM refineries show a good effluent wastewater quality indicating more complete treatment facilities.

Table B-IV-B-10 shows the results of applying the expected removal efficiencies of the high degree of treatment processes to the total effluent of the typical refinery (10 MGD wastewater discharge). Also listed are the representative wastewater quality parameters of the typical C-SELM refinery based on discharge records. The quality of effluent from C-SELM refineries indicates a high degree of treatment with possible carbon polishing for phenol wastes. Heat loads are computed based upon communications with the American Petroleum Institute.[9] The heat required for the refining of one barrel of crude oil has been estimated to vary between one-half and one million BTU. About 25% of this energy is dissipated in cooling water. For once-through cooling systems, an approximate increase in temperature of 40°F is indicated for the cooling water.[9]

The heat remaining in typical refinery effluents prevalent in areas other than C-SELM was computed by assuming that cooling waters were cooled on towers and recirculated to reduce intake and discharge from 44 MGD to 10 MGD with a 20°F approach temperature for the cooling tower.

The effluent qualities presented in Table B-IV-B-10 may be compared to the water quality standards of Indiana which are tabulated in Table B-IV-B-11. Phenols in C-SELM refinery effluents are about 15-times the allowed concentration, neglecting any possible dilution factor. Since carbon adsorption of phenols is the most complete treatment available today, the extent of present treatment in this area is satisfactory.

Table B-IV-B-9

DEGREES OF PRESENT-DAY TREATMENT

Low Degree of Treatment

1. API Separator
2. Slop Oil Treatment

Intermediate Degree of Treatment

1. API Separator
2. Aerated Lagoon
3. Slop Oil Treatment
4. Sour Water Stripping

High Degree of Treatment

1. API Separator
2. Activated Sludge
3. Thickening and Vacuum Filtration (Sludge)
4. Sludge Incineration
5. Slop Oil Treatment
6. Sour Water Stripping

Table B-IV-B-10

100,000 bpsd TYPICAL REFINERY

EFFLUENT QUALITY OF TOTAL EFFLUENT AFTER PRESENT TREATMENT

P A R A M E T E R

Refinery	Flow	Oils	Chloride	Sulfide	BOD₅	Phenols	Ammonia	Total Phosphorus	Suspended Solids	Total Dissolved Solids	Heat
Typical - High Degree of Treatment Other than C-SELM Area	10 MGD	-	-	50# 0.6 mg/l	500# 6 mg/l	100# 1.2 mg/l	-	-	--	--	2×10^9 Btu
Typical - High Degree of Treatment C-SELM Area	44 MGD	1850# 5 mg/l	Overly variable (5 to 8000 mg/l)	Trace 0	3700# 10 mg/l	10# 0.03 mg/l	5 mg/l	0.2 mg/l	5500# 15 mg/l	7350# 200 mg/l	14.7×10^9 Btu

Table B-IV-B-11

CURRENT WATER QUALITY STANDARDS - INDIANA - INNER HARBOR REGULATIONS
(Parameters Relating to Refinery Discharges Only)

Concentration Units, mg/l

Ammonia	Chlorides	Sulfides	Phenols	Oils	Total Dissolved Solids	Suspended Solids	Temperature [a]
0.05	20	No Standard	0.002	None visible (15 mg/l)	197	No increase above background concentration	
					167[c]	8[c]	

Month	Max. Temp-°F	Ambient-°F [b]
January	45	40
February	45	35
March	45	40
April	55	45
May	60	52
June	70	58
July	80	63
August	80	68
September	80	66
October	65	63
November	60	55
December	50	46

[a] At any time and at a maximum distance of 1,000 feet from a fixed point adjacent to the discharge and or as agreed upon by the State and Federal regulatory agencies, the receiving water temperature shall not be more than 3 degrees Fahrenheit above the existing natural water temperature nor shall the maximum temperature exceed those listed below whichever is lower.

[b] Assumed Ambient Lake Michigan Temperatures.

[c] Reported Lake Michigan Intake Water Quality.

Heat quantities must be reduced if present C-SELM refinery discharges are to meet closed-cooling-system standards for Indiana. This will be discussed in the next section on treatment to meet current standards.

Treatment required to satisfy current standards. As is stated above, C-SELM area refineries will require closed-loop cooling systems to satisfy current temperature standards. With the assumption that total dissolved solids will be the limiting factor after ten cycles, a cooling-recirculation scheme is applied to the system which reduces intake flows from 44 to 5 MGD. The same cooling tower design parameters are used for the reference refineries outside the C-SELM area. The heat reduction due to the cooling towers is about 92%, so that effluent heat amounts to only 1.2×10^9 BTU/day. Due to the suspended solids concentration in present C-SELM effluent, there is a need to remove suspended solids from a closed cooling-recirculation system. Background suspended solids concentrations are approximately 8 mg/l, and concentration through recycling will require reductions to background levels. This requirement is met by using high rate pressurized filters prior to the cooling towers. Removals of suspended solids will lower, considerably, the effluent oil concentrations and should further reduce BOD_5[4]/ Figure B-IV-B-6 shows one possible flow diagram for this system. A one-MGD portion of cooling tower blowdown is recycled to partially satisfy process water demands. Post-areation has been added following the carbon columns due to the low oxygen content of the treated effluent.

The reduction of the volume of some streams by as much as ten-times causes some problems in reevaluating the existing treatment facilities for operational adjustment and applications. Exact capacity requirements and operational modes would require field and experimental data from the specific refinery in question. From past experience it is known that by concentrating the load through volume reduction, the cost of a treatment facility generally is reduced. The operation and maintenance costs of the activated sludge and sludge handling processes, the carbon columns and the chlorination facilities were reduced in direct proportion to the reduction in flow. Economy of scale was applied, however, which increased the unit costs of operation. The carbon column operation was altered from biological type to physical-chemical type.

Treatment required to satisfy NDCP. Table B-IV-B-6 lists the effluent quality expected for NDCP standards and Figure B-IV-B-7 shows a possible treatment sequence to meet NDCP standards. A comparison of Tables B-IV-B-10 and B-IV-B-6 shows that, in terms of ammonia, phosphorus and BOD_5 concentrations, effluents meeting current standards will require further treatment in order to satisfy NDCP standards, especially after the concentrating effects of the cooling water recycle system. A one-MGD portion of cooling tower blowdown
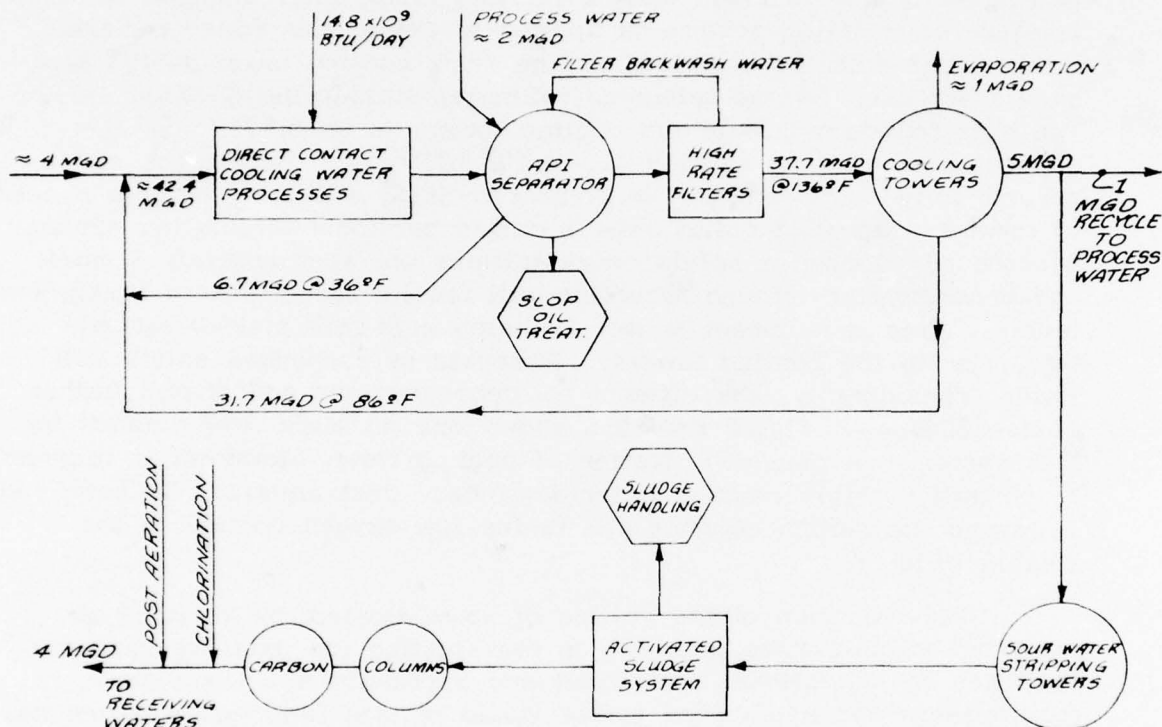
Figure B-IV-B-6

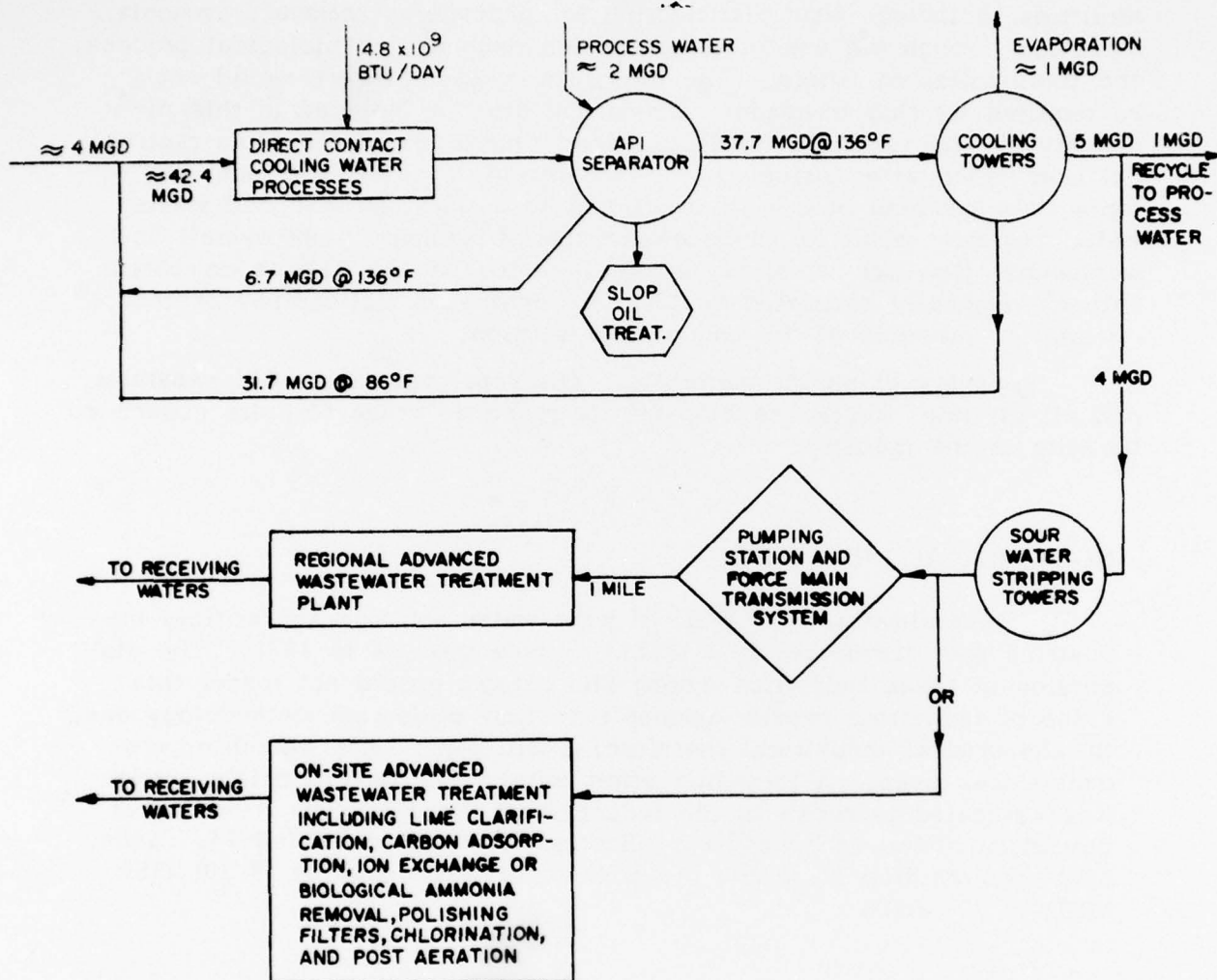POSSIBLE FLOW DIAGRAM OF TREATMENT FACILITIES
TO MEET CURRENT STANDARDS

B-IV-B-31

Figure B-IV-B-7

POSSIBLE FLOW DIAGRAM OF TREATMENT FACILITIES
TO MEET NDCP STANDARDS

is recycled to partially satisfy process water demands. Two NDCP alternatives are considered. The first would expand on-site treatment facilities to include lime clarification for phosphorus removal, ammonia reduction through the use of either an ion exchange or biological process, and final polishing filters. The high rate pressure filters would not be required for this treatment scheme and are not included in this alternative. The second alternative would require on-site cooling-recirculation, sour water stripping, API separation, slop-oil treatment and force main pumping of the plant effluent to a regional advanced wastewater treatment plant for physical-chemical treatment. The overall unit wastewater flow per barrel of petroleum is reduced by 91% as compared with an originally estimated 92.5%. An extensive bibliography of source material is provided at the end of this section.

Recovery of usable materials. Oil separated in the API separator and all oil later treated as slop-oil for reuse as crude oil, are economic benefits to the industry.

NON-CRITICAL INDUSTRIES

Approximately 350 MGD of wastewater not from the critical industries was discharged to C-SELM surface streams in 1970. The dispersion of these industries among SIC categories did not render this group of industries readily amenable to flow projection methodology used for the critical industries; therefore, a different, more workable, procedure was used. A recycling effort within the non-critical industries was estimated to result in the reduction of the 350 MGD flow to that proportion shown in Appendix B, Section III-B, page B-III-B-14. Thus, by 2020, the flow from this group of industries would be (0.20) (350 MGD) = 70 MGD.

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IV.   COMPONENT BASIS OF DESIGN

C.   SLUDGE MANAGEMENT SYSTEMS

INTRODUCTION

The Summary of Design for sludge management systems describes the types of sludges generated by the C-SELM treatment system alternatives, the beneficial utilization objectives, and the management techniques necessary to accomplish these objectives. In this section these same subjects are discussed in more detail.

SLUDGE TYPES

Two types of sludge resulting from the treatment of wastewater are biological sludge and physical-chemical sludge. Biological sludges contain the organic matter found in raw wastewater either in its natural form or in another form resulting from the treatment process. Physical-chemical sludges contain the largely inorganic residue remaining after the incineration encountered during lime recalcination.

Biological sludges are further classified into three types: conventional biological; advanced biological; and land treatment. The characteristics and yield of each type are described here.

There is only one type of physical-chemical sludge considered in this study. This sludge is also evaluated for its characteristics and yield.

The following sub-sections discuss in detail the results of these examinations.

Biological Sludges

Conventional and land treatment sludges. These sludge types are discussed together because they exhibit similar characteristics and yields. Conventional and land treatment sludges contain high concentrations of decomposable organic matter or volatiles which can be stabilized to prevent the production of offensive odors. Anaerobic digestion is the process selected for this stabilization and a detailed discussion of its design basis may be found in Appendix B, Section IV-A. Thickening in lagoons is subsequently provided to insure that

the digested conventional biological sludge will attain a 6% total solids concentration.

The yield of these sludges (including grit) is expected to be 0.77 dry tons of digested sludge per million gallons of sewage inflow. This value was obtained using the calculations presented in Data Annex B, Section IV-C.

The chemical constituents and the characteristics and yields of these sludges are summarized in Table B-IV-C-1 and Table B-IV-C-2, respectively.

Advanced biological sludge. This type of sludge also contains a significant concentration of decomposable organic matter or volatiles, which are also stabilized by anaerobic digestion. It also contains the calcium compounds resulting from the lime recalcination process which is part of the advanced biological treatment system. The mixture of these two materials is assumed to be handled at a 6% total solids concentration.

The yield expected for advanced biological sludge is 1.64 dry tons of digested sludge and solids per million gallons of sewage inflow. The computations used in obtaining this value are presented in the Data Annex B, Section IV-C.

The chemical constituents and the characteristics of this sludge are summarized in Table B-IV-C-1 and Table B-IV-C-2, respectively.

Physical-Chemical Sludge

During incineration in the lime recalcination process, which is an integral part of physical-chemical treatment technology, the raw physical-chemical sludge is converted into a dry inorganic material composed of ash and calcium compounds. It does not have the valuable nitrogen and organic nutrient content of the biological sludges.

This white material has a fine powdery texture which could cause serious dust control problems when handled or transported in the dry state. To avoid these problems and to facilitate economical pipeline transportation, water is mixed with the inert solids to form physical-chemical sludge containing no more than 10% total solids by weight. This concentration allows the sludge to be easily conveyed by pipeline with reasonable pump maintenance.

Table B-IV-C-1.

CHARACTERISTICS OF WASTE SLUDGES AND LAND APPLICATION RATES[c]

Types of Sludge	Yield Dry Tons/MG	% Solids for Pipeline Transmission[a]	Agricultural Application Rate Dry Tons/Ac/Yr.	Accumulation in 50 Years Dry Tons/Acre[b]	Land Reclamation Application Rate Dry Tons/Acre	Accumulation in 50 Years Dry Tons/Acre
Advanced Biological	1.64	6	28.8	1,440	213	213
Chemical-Physical	1.13	10	1.73	86.5	-	-
Conventional Biological	0.77	6	13.5	675	100	100
Land Treatment	0.77	6	13.5	675	100	100

[a]These sludges are all amenable to greater dewatering. This would be appropriate for laternative transportation systems such as unit train or barge transport.

[b]The amounts are believed to be acceptable from the point of view of accumulations of heavy metals which accompany all sewage sludges. These metals are kept largely insoluble by maintaining a pH of about 7.

Table B-IV-C-2

CHEMICAL CONSTITUENTS OF SLUDGES
(Percentage by Weight)

Constituent	Conv. Biolog. & Land Treat.	Advanced Biological	Physical- Chemical
Ash	51.7	24.1	19.
Volatiles	48.3	24.5	0.
$Ca_5(OH)(PO_4)_3$	0.	11.2	21.
$CaCO_3$ [a]	0.	40.2	60.

[a]Calcium compounds presented in terms of a calcium carbonate equivalent.

The yield of physical-chemical sludge is expected to be 1.13 dry tons per million gallons of sewage inflow. This value is based on computations presented in the Data Annex B, Section IV-C.

The chemical constituents and the characteristics and yield of physical-chemical sludge are summarized in Table B-IV-C-1 and Table B-IV-C-2, respectively.

SLUDGE UTILIZATION OBJECTIVES

General

The spreading of sludge on and into the soil provides for a beneficial utilization of the C-SELM sludge. Biological sludges are rich in minerals and nutrients vital to crop growth. This same valuable mineral and nutrient resource makes biological sludge an important source of soil-building material for the reclamation of heretofore agriculturally unproductive or sterile land. Furthermore, the incorporation of this sludge in the land returns its nutrient and mineral contents once again to food production, thereby relieving demands on fertilizer production and on energy and mineral resources required to produce this fertilizer.

The sludge generated as a by-product of the physical-chemical treatment system is chemically inert and sterile. While this sludge has no nutrient or humus value, it can be used agriculturally for soil pH control and as a soil conditioner. It has a high alkalinity caused by the alkaline calcium compounds in its composition which can be used advantageously. Many agriculturally useful C-SELM areas require a means for soil pH control. Physical-chemical sludge can provide this control quite easily with regulated applications to those areas needing an increase in soil pH.

Alternative utilization objectives for biological sludges include drying for fertilizer production, wet oxidation for disposal, and fermentation for the manufacture of animal feeds. Present practice in C-SELM indicates that drying and wet oxidation are not cost-competitive with the current practice of wet agricultural applications to land. Production of animal food from sludges is a desirable and potentially practical use for sludge, but is still in a developmental stage and has not yet been proven to be economically competitive.

Two specific utilization objectives for which sludge can be applied to the land have been developed for the different C-SELM sludges. These are agricultural utilization and land reclamation.

The following sections discuss these two application objectives with respect to their overall concepts, their sludge application rates, and their potential locations in the C-SELM region. A detailed discussion of the modular basis of design for these two land application techniques is found in the following sub-section, Sludge Management Techniques.

Agricultural Utilization

The agricultural utilization of sludge employs sludge as a fertilizer or a soil conditioner. The land selected for this type of application is used continually for agricultural purposes. The sludge is applied to the agricultural land at a controlled rate on a yearly basis. The sludge application system is by necessity a permanent installation. Both biological and physical-chemical sludges are used for this type of sludge utilization objective.

Application rates. The biological sludge application rates for agricultural utilization are assumed to be controlled by the nitrogen balances in each system. The actual nitrogen removal mechanisms for the C-SELM system design are: crop uptake, biological denitrification and ammonia volatilization. A large amount of the nitrogen found in the biological sludges is in the organic form and is not readily available for removal by the mechanisms listed above. Mineralization of this organic nitrogen to ammonia nitrogen, however, occurs under steady state conditions thus allowing the subsequent processes of ammonia volatilization and nitrification to take place. The nitrate nitrogen produced during nitrification then is partly removed by crop up-take in the root zone and partly removed by bacterial action, denitrification, in which the nitrates are transformed to nitrogen gas which escapes to the atmosphere.

The following assumptions are used in the determination of the biological sludge application rates for both agricultural utilization and land reclamation:
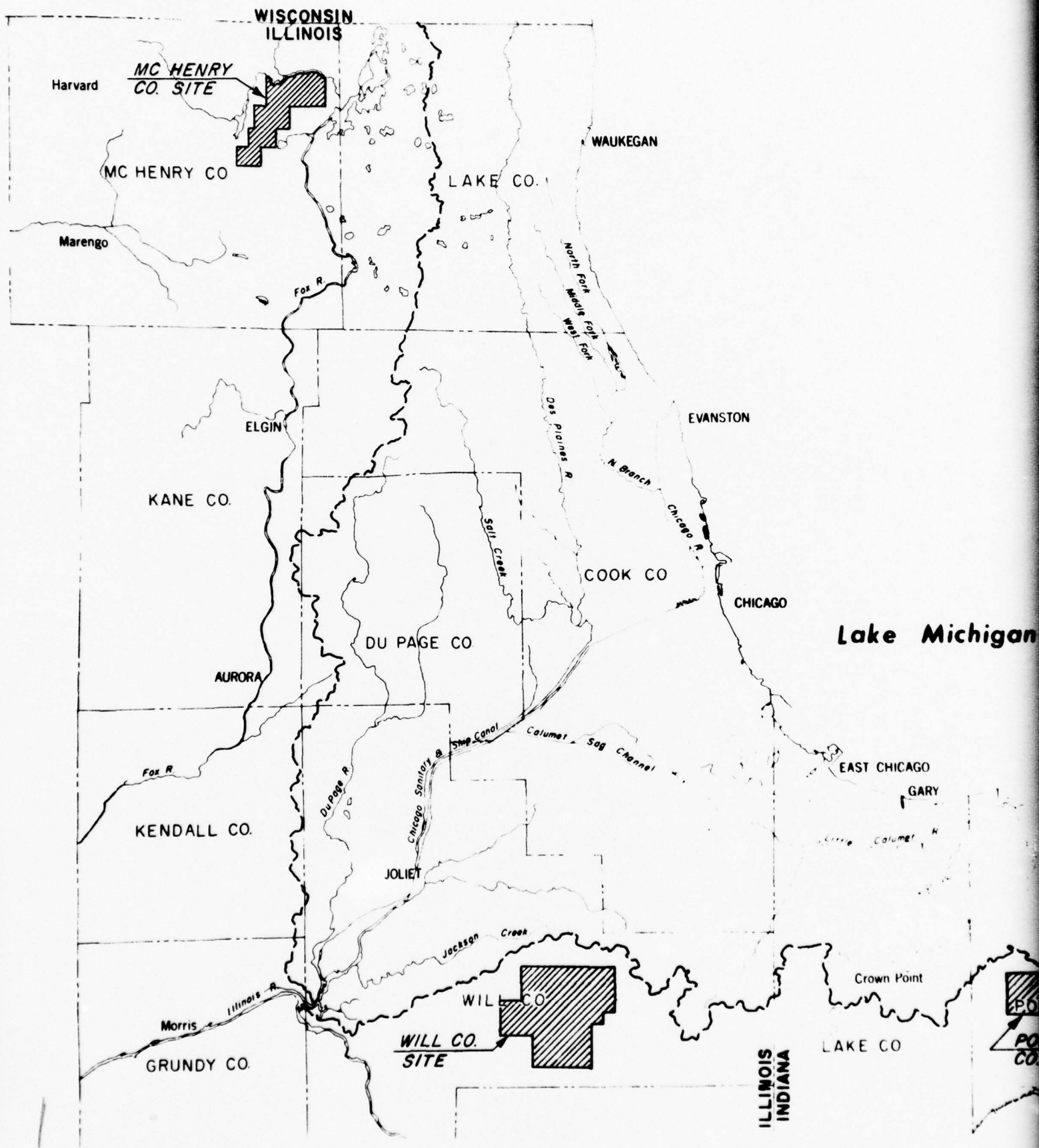
1. Nitrogen comprises 4% of the sludge total solids. Of this nitrogen, one-third is in the ammonia form and two-thirds are in the organic form.

2. With the ammonia stripping process (land reclamation option only), only 25% of the ammonia nitrogen will remain as a residual nitrogen.

3.   Approximately 4% of the available organic nitrogen remaining
in the soil mineralizes to ammonia nitrogen each year.

4.   For agricultural applications, the yearly volatilization loss of
ammonia nitrogen is approximately 50% of the applied ammonia
nitrogen.   For land reclamation applications, the volatilization
loss of ammonia nitrogen is approximately 50% of the applied
ammonia nitrogen remaining after ammonia stripping.

5.   Denitrification accounts for approximately 30% of the nitrogen
yearly available to the crop as nitrate nitrogen.

6.   No accumulation of organic nitrogen occurs by means of na-
tural soil-building processes.

7.   The yearly rate of nitrogen uptake by a grass crop is 360
pounds of nitrogen per acre per year.

        Based on the above assumptions, the desirable application of
total nitrogen in the agricultural utilization of biological sludge is
computed to be 515 pounds/acre/year.   Using this value, the design
application rates for each type of biological sludge are computed.
Computations are given in data Annex B, Section IV-C, and sludge
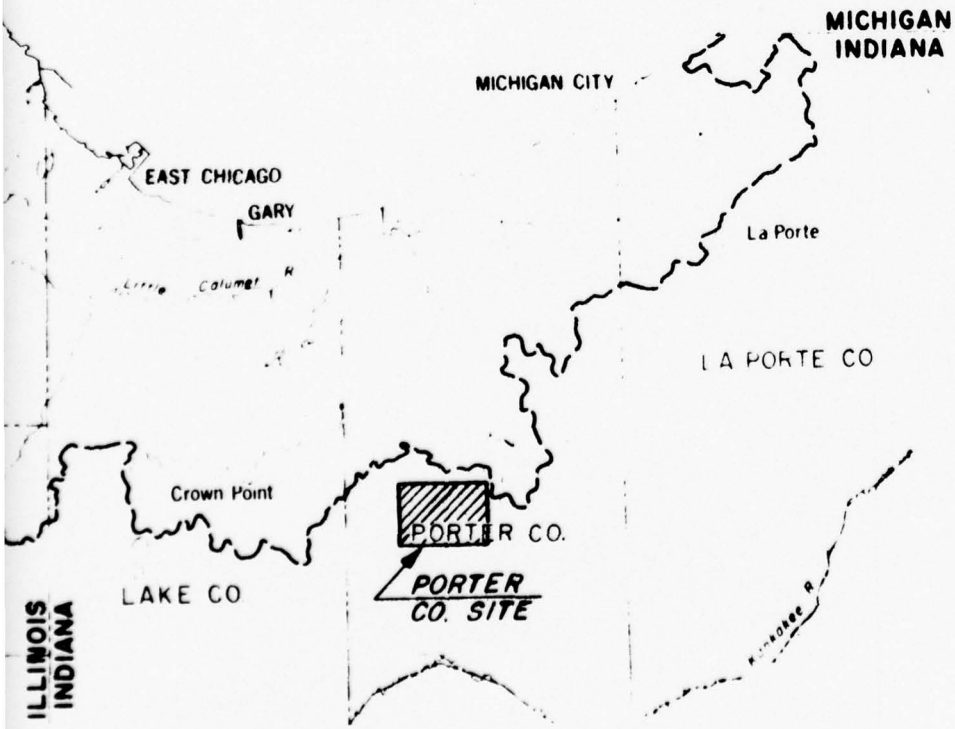applications rates are presented in Table B-IV-C-1.

        Since physical-chemical sludges do not contain significant
amounts of nitrogen, their application rates do not depend on the
nitrogen balance in the soil-sludge system.   Instead, the physical-
chemical sludges are applied to the land at a rate which is equiva-
lent to an application rate of calcium carbonate of one dry ton/acre/
year.   This application rate, shown in Table B-IV-C-1, is consistent
with current agricultural practice in central and northern Illinois for
the liming of clay soils. [1]/

        Land sites and management.   The potential sludge application
sites for agricultural utilization of conventional, advanced biological,
and physical-chemical sludges are the rural areas adjoining the C-SELM
study area.   These sites are located in McHenry County and Will
County, Illinois and in Porter County, Indiana, respectively, and are
typically shown in Figure B-IV-C-1.   The potential sludge application

WISCONSIN
ILLINOIS

Harvard

MC HENRY CO. SITE

MC HENRY CO

Marengo

Fox R.

LAKE CO.

WAUKEGAN

North Fork
Middle Fork
West Fork

Des Plaines R

EVANSTON

N Branch

Chicago R.

ELGIN

KANE CO.

Salt Creek

COOK CO

CHICAGO

DU PAGE CO

Lake Michigan

AURORA

Fox R.

DuPage R.

Chicago Sanitary & Ship Canal

Calumet - Sag Channel

EAST CHICAGO

GARY

KENDALL CO.

Little Calumet R.

JOLIET

Jackson Creek

Crown Point

Illinois R.

Morris

WILL CO.

WILL CO. SITE

PO.
CO.

GRUNDY CO.

ILLINOIS
INDIANA

LAKE CO

LEGEND

LOCATIONS OF POTENTIAL
AGRICULTURAL UTILIZATION SITES

Lake Michigan

MICHIGAN
INDIANA

MICHIGAN CITY

La Porte

EAST CHICAGO

GARY

Little Calumet R

LA PORTE CO

5   0   5

MILES

Crown Point

PORTER CO.

Figure B-IV-C-1

PORTER
CO. SITE
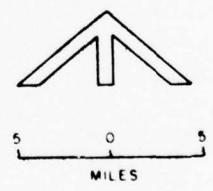
POTENTIAL SLUDGE DISPOSAL SITES
FOR AGRICULTURAL UTILIZATION

LAKE CO

ILLINOIS
INDIANA

B-IV-C-8

sites for agricultural utilization of land treatment sludges are adjacent to the land treatment storage lagoons. The locations of these sites are given with the overall land treatment site locations in Appendix B, Section IV-A.

All sites selected for the agricultural utilization of sludge are in existing agricultural areas where the humus content of the soil is already sufficient to sustain crop growth. A 90 percent land utilization of presently tilled ground is assumed in determining the gross sludge acreage requirements. The 10 percent remaining land allows for farm buildings, private farm roads, stormwater runoff control, etc.

The land required for the agricultural utilization of sludge could be secured by a variety of mechanisms, including purchase, purchase with lease-back and contract. As an illustration and for the purpose of the C-SELM study, a contract is envisioned between the landowner and the Regional Wastewater Management System that provides for their separate and mutual interests over both a short and long-term time period. The owner would guarantee the farming of the land in a manner consistent with the goals of the Regional Management System for a period of 50 years. In exchange for this commitment by the owner, the Regional Wastewater Management System would install the necessary sludge distribution and application capital equipment and would pay an initial and annual fee. The initial fee would recognize the potential loss of crops to the owner during the installation of sludge management equipment and the farming constraints imposed on the owner by the sludge management system. The annual fee would recognize the potential loss to the present or future owner from holding the land in agricultural land use as contrasted with more valuable land uses.

The biological sludges would be applied to grass-type crops on a one-application-per-year basis and for an application season of eight months. The harvesting of the grass crop would be by cutting at approximate three-week intervals during the growing season. The harvesting schedules would anticipate the fertilization, or sludge application, schedules and would provide for a cutting just before the interruption in the growing schedule caused by the sludge application system. The grass crop would subsequently reestablish its root system and begin to thrive again as a harvestable crop. Reed canarygrass is the grass crop selected for illustration in the C-SELM agricultural utilization or biological sludge system. A description of reed canarygrass is presented in Data Annex B, Section IV-A. The owner retains ownership of the grass crop under the terms of the con-

tract with the Regional Wastewater Management System. This grass-growing-agricultural-sludge-management system, combined with a corn-producing-land-treatment technology for wastewater reclamation, creates the opportunity for integrated livestock production in proximity to the C-SELM urban area. Large feed lots would be involved with feed lot wastes being returned for fertilization to the interstitial non-irrigated agricultural land associated with the land-treatment agricultural land.

Alternative agricultural-biological-sludge-management systems can be contemplated which produce grain crops. While such sludge management systems have the advantage of producing a more readily marketable grain product, they are somewhat more costly management systems because of the relatively short time during which sludge fertilizer may be applied without interfering with the grain crop.

The physical-chemical sludges would be applied to virtually any type of crop on a one-application-per-year basis, for an application season of eight months. The acidity-controlling inert sludge would be applied to the soil by a controlled spray technique with minimum effect on the crop.

The owner would supply his own fertilizer and would retain ownership of crops.

Land Reclamation

The land reclamation utilization of sludge involves the application of sludge to strip-mined areas to build humus content and fertility in the soil. As the soils in strip-mined areas contain only small quantities of organic matter or humus, those soils are often devoid of plant life. The land reclamation system used as an illustration for the C-SELM study anticipates application of sludge over a short period of time to correct these soil deficiencies and to stimulate the growth of grass or trees for recreational purposes. As this application is an infrequent operation, the sludge application system installed in a land reclamation site is temporary. Only biological sludges are used for land reclamation application as physical-chemical sludge does not contain nitrogen or have any humus value.

Application rates. The biological sludge application rate for land reclamation is assumed to be controlled by the system nitrogen balance in a way similar to that already discussed for agricultural utilization. The primary difference between the two systems is that ammonia stripping, or some other form of control of excess nitrogen, will be used with land reclamation so that a larger short-duration application of sludge can be made without excessive amounts of nitrogen being transmitted to streams. This can be done because, after re-

moving most of the ammonia nitrogen in the sludge, the remaining nitrogen is primarily in the organic form and the rate of mineralization of organic nitrogen is low. Thus, the conversion of organic nitrogen to ammonia nitrogen will take place slowly and the high-rate, short-duration application of large amounts of sludge will not produce any problems with nitrogen pollution of the groundwater or of the surface water.

To provide for this removal of excessive ammonia nitrogen in the sludge, lime is added to the sludge to elevate the pH of the sludge to a point where the ammonia dissolved in the sludge can be air-stripped as the sludge is sprayed through the air. The optimum pH values and the corresponding lime dosages for ammonia stripping have been determined in laboratory studies, and the results presented in Data Annex B, Section IV-C, show a pH of 11 to be required for 75% ammonia removal.

The land reclamation application, accompanied by ammonia stripping, is advantageous in areas where the soil and associated soil moisture and acid content exists, as for example, in the acid mine waste areas of Illinois and Indiana. The excess lime associated with the sludge helps to mitigate the unnaturally high acidity caused by the acid-providing elements present in these coal spoil areas. In other areas of Illinois and Indiana, coal mining has produced strip-mined areas without the associated surface acidity conditions. For these latter strip-mined areas, there now exists evidence that a single or infrequent land reclamation application of biological sludge, unaccompanied by ammonia stripping, can be made without deleterious effects on groundwater. [2/] Surface runoff is always managed on-site for any sludge management system to prevent possible deleterious effect on surface water quality.

The assumptions listed for the determination of sludge application rates for agricultural utilization are also used in determining the rates of sludge application for land reclamation, and will not be listed again. Based on these assumptions, the permissible application of total nitrogen for a grass crop is 515 pounds/acre/year. Using this value, the optimum single application for land reclamation, using the advanced biological and land treatment sludges, are estimated to be 213 and 100 dry tons per acre, respective. These values are also given in Table B-IV-C-1. The difference in application rates for these sludges is due to the difference in their nitrogen contents. Conventional biological sludge can also be used for land reclamation purposes, but as conventional treatment alone has not been considered as a final wastewater management alternative, its use for purposes of land reclamation was not considered.

Land sites and management. The huge existing or potential strip-mined areas located in Knox County and Fulton County, Illinois and in Clay County, Indiana were selected as possible sludge application sites for land reclamation. They are shown in Figure B-IV-C-2. The information on the location and size of these possible land reclamation sites was obtained from the Midwest Coal Producers Institute and the State of Indiana Department of Mines. For the planning purposes of the C-SELM study, it is assumed that the land for these sites is not purchased. Based upon exploratory discussions with the coal producers, the land reclamation application of sludge would be provided to the coal landowners in exchange for free access to the land. The sludge distribution and application systems would be supplied by the Regional Wastewater System.

SLUDGE MANAGEMENT TECHNIQUES

The presented component basis of design for sludge management techniques is presently partly in this section and partly in Appendix B, Section IV-A. Those parts of the sludge management systems which deal with sludge digestion, sludge incineration or recalcination, sludge thickening and sludge storage at the treatment plant are discussed in Appendix B, Section IV-A. This section will deal with the transportation of sludge to the land application sites and with the modular design for the land utilization of sludge

Transportation

Four modes of transportation, truck, train, barge, and pipeline, are available for the transportation of sludge from its generation site to its application site. Truck and pipeline systems can be operated independently while train and barge system require an auxiliary mode of transportation due to the fixed routes of these systems. In most cases, an auxiliary transportation system such as a pipeline can transport the sludge to or from the existing rail line or waterway.

Pipeline system. Pipeline systems can be applicable for the transportation of sludge to all sludge utilization areas considered in the C-SELM study. A pipeline system used for the transportation of sludge, is a combination of one or more sub-systems. Each subsystem includes a pumping station and a pipeline. The pumping station has a wet well for flow rate adjustments, a dry pit for pump motors, the necessary control system installation, protection from adverse weather conditions, and proper ventilation.
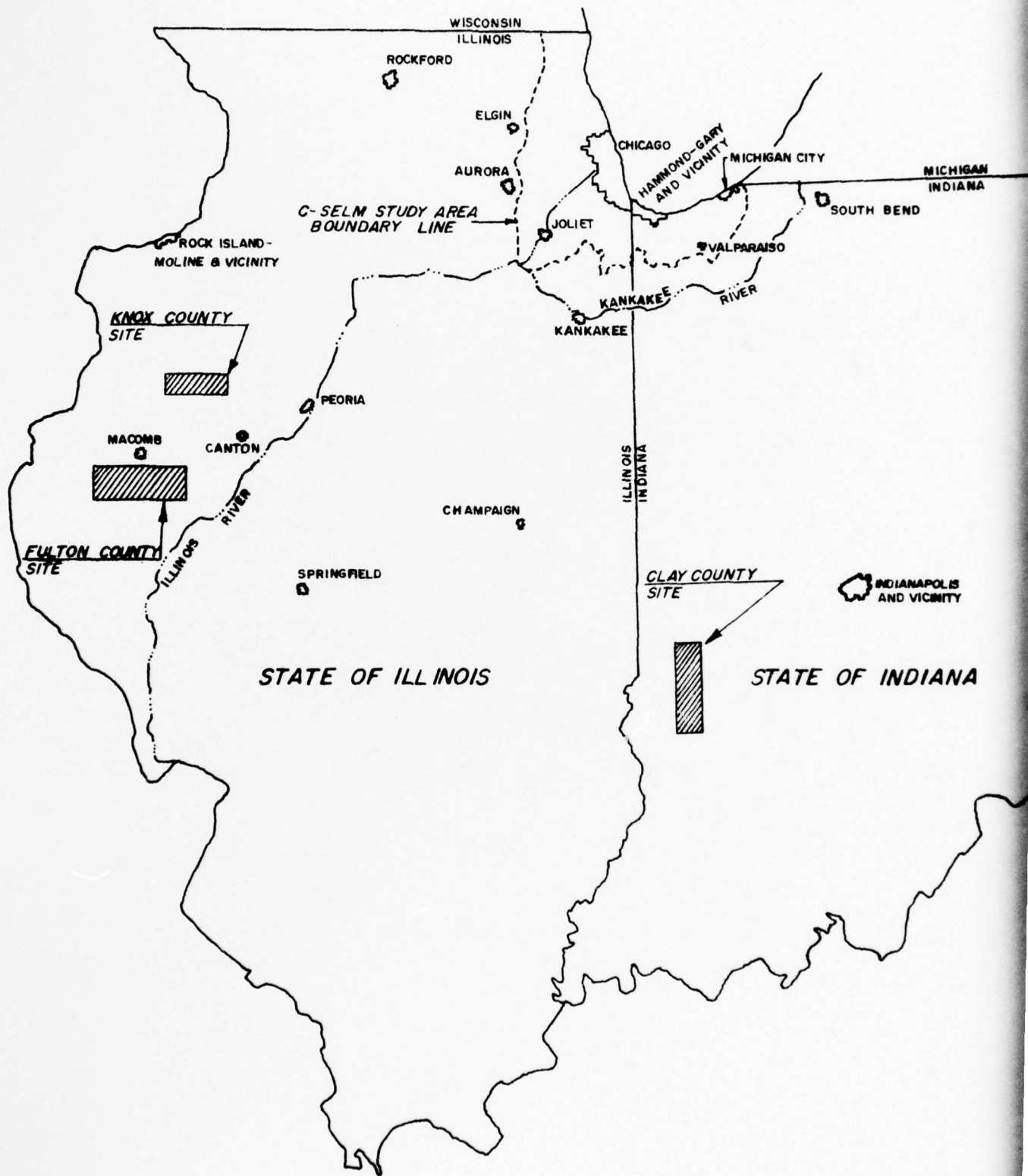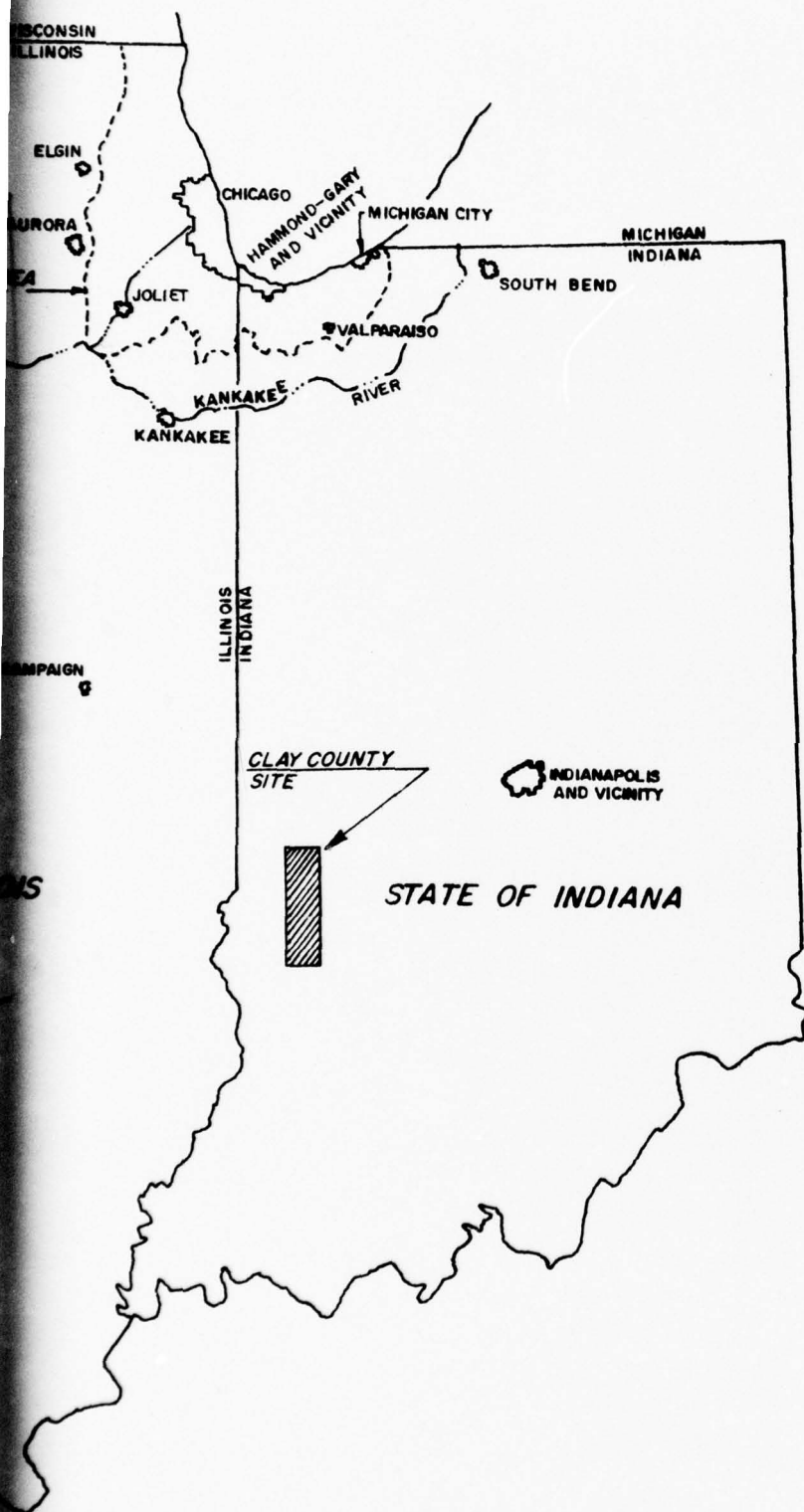
LEGEND

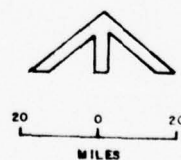LOCATIONS OF POTENTIAL LAND
RECLAMATION SITES

Figure B-IV-C-2

POTENTIAL SLUDGE DISPOSAL SITES
FOR LAND RECLAMATION

B-IV-C-13

At least three pumps are installed in each pumping station, including one stand-by pump. Pumps are of the centrifugal type with a pumping head of 200 feet or more and an efficiency of greater than 40%. The pumping station is provided with sufficient maintenance equipment and space to insure proper and efficient maintenance. An emergency mobile Diesel generator is provided for each pipeline system. This Diesel generator could be used for all of the sub-systems in a pipeline system.

Cast iron or concrete pressure pipe with a glass lining is used for the pipeline. The pipe is sized to maintain a minimum velocity of two feet per second at design flow and the friction head is computed by using a Hazen-Williams coefficient of 60. The pipe is designed with a thickness able to resist the static pressure and the surge pressure expected under normal operating conditions and under transient conditions as well.

Barge system. For a barge transportation system, the sludge generated in the C-SELM study area is first collected at a barge loading point by means of pipeline system. This collection point is located at the existing sludge lagoons near the Stickney sewage treatment plant of the Metropolitan Sanitary District of Greater Chicago (MSD).

Four barges with a capacity of 1,200 wet tons of sludge each are used in conjunction with a towboat to transport sludge to the application site. The average speed of the barge is five mph, and the barges are loaded and unloaded by an auxiliary pipeline system with a loading or unloading time of eight hours.

This transportation system can be applicable to the transportation of sludge from the C-SELM study area to the Fulton County and Knox County, Illinois strip-mining areas via the Chicago Sanitary and Ship Canal and the Illinois River.

Railroad system. For a railroad transportation system, the sludge generated in the C-SELM study is first collected at a railroad loading point by means of a pipeline system. This collection point is located at the existing sludge lagoons, again near the MSD Stickney sewage treatment plant.

Each sludge train is composed of 40 or more tank cars, each with a capacity of 20,000 gallons. The average train speed is 20 mph. The sludge is loaded and unloaded with a manifold pipeline system. The loading or unloading time is six hours.

This transporation system can be applicable for the transportation of sludge from the C-SELM study area to the Fulton County and Knox County, Illinois strip-mining areas via the Atchison, Topeka, and Santa Fe Railroad.

Truck system. A truck system can accomplish two functions, collection of sludge in the C-SELM study area and transportation of sewage sludge from the C-SELM study area to a land application area. Each truck can have a capacity of 6,000 gallons and might travel at an average speed of 15 mph in an urban area and 30 mph in a rural area. The loading or unloading time is 30 minutes and the transported sludge has a 6% solid content by weight.

This transportation system can be applicable in short haul situations where the sludge application sites are reasonably close to the sluge collections points.

Application To Land

Application systems for both agricultural utilization and land reclamation are designed as modular systems to be used for different sludge types and at different application sites without changing the basic layouts shown in Figure B-IV-C-3 and Figure B-IV-C-6. Table B-IV-C-3 gives a summary of possible sludge management options for the different sludge types along with their potential application sites.

Agricultural utilization. A modular system for the agricultural utilization of sludge is designed using the following operation of components. The sludge transported to the land application area is first stored in a storage lagoon with a ten-day storage capacity. A pumping station is located near the storage lagoon and is connected to the storage lagoon by means of an open channel which could also provide additional storage. After storage, the sludge is distributed by a system composed of the pumping station, header pipes, and lateral pipes. Their arrangement is shown in Figure B-IV-C-3. Each lateral pipe has ten quick-coupling units, evenly distributed over the entire length of the pipe.

For the biological sludge a moldboard plow, as shown in Figure B-IV-C-4, is connected to a quick coupling by a flexible hose. A single plow is served by one pair of laterals on both sides of the header pipe. Each plow can complete one cycle of application in an eight-month application season per year. The moldboard moves for-
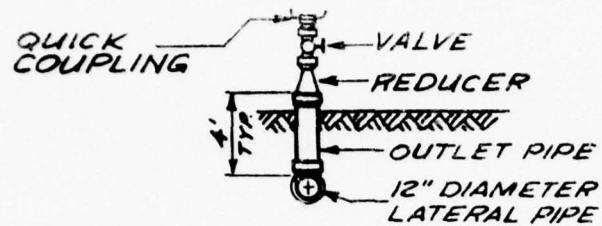
DETAIL OF QUICK COUPLING
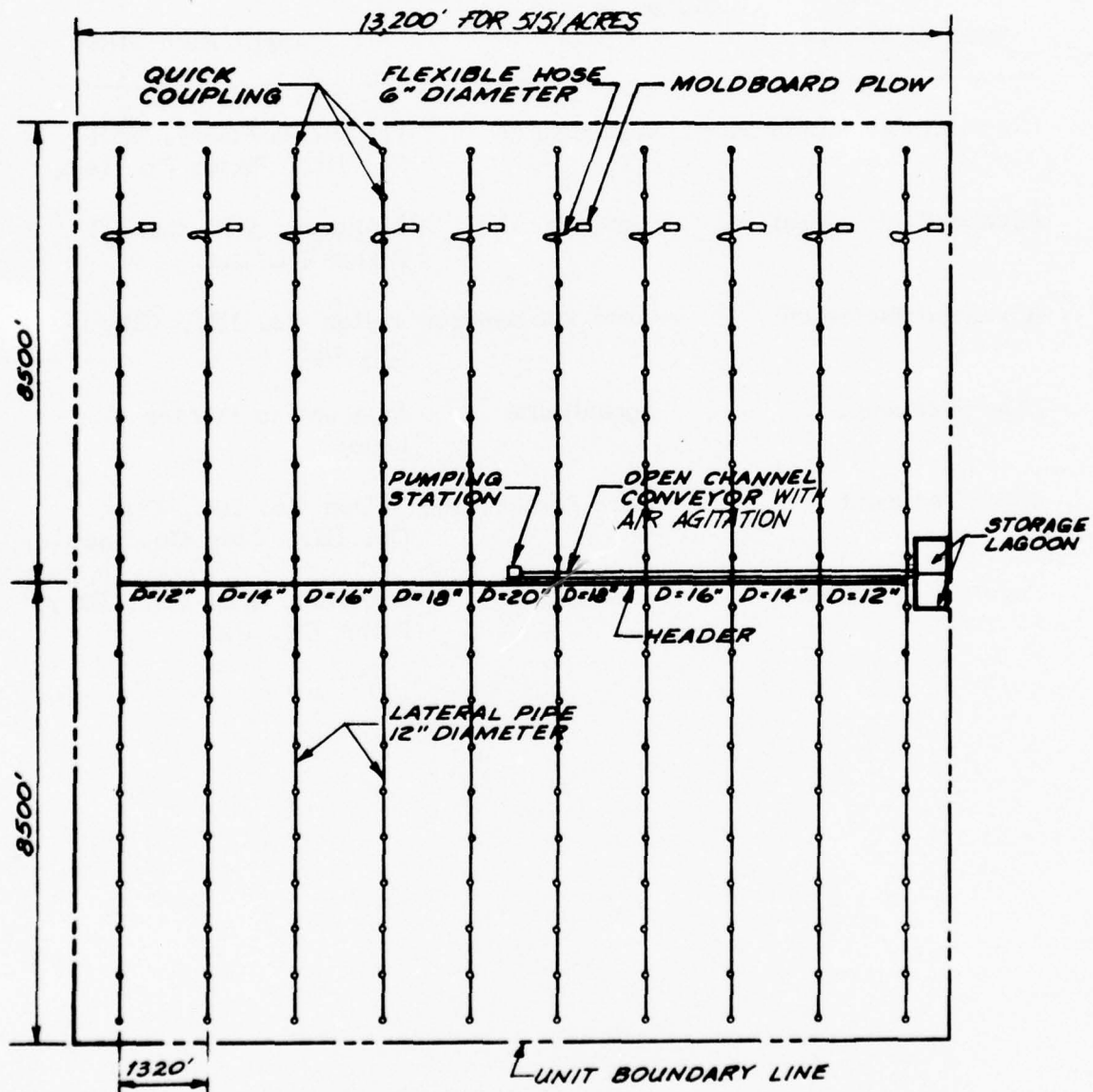
Figure B-Ⅳ-C-3

TYPICAL DISTRIBUTION SYSTEM FOR REGULAR
APPLICATION OF ADVANCED BIOLOGICAL SLUDGE

Table B-IV-C-3

SLUDGE MANAGEMENT OPTIONS

Type of Sludge	Sludge Management Option	Application Site
Conventional Biological	Agricultural	Fulton, McHenry, Will Co. Ill., Porter Co. Ind.
Advanced Biological	Agricultural	McHenry, Will Co. Ill. Porter Co. Ind.
Advanced Biological	Land Reclamation	Fulton Co. Ill., Clay Co. Ind.
Land Treatment	Agricultural	Adjacent to Storage Lagoons
Land Treatment	Land Reclamation	Fulton Co. Ill., Clay Co. Ill., Clay Co. Ind.
Physical-Chemical	Agricultural	McHenry, Will Co., Ill., Porter Co. Ind.
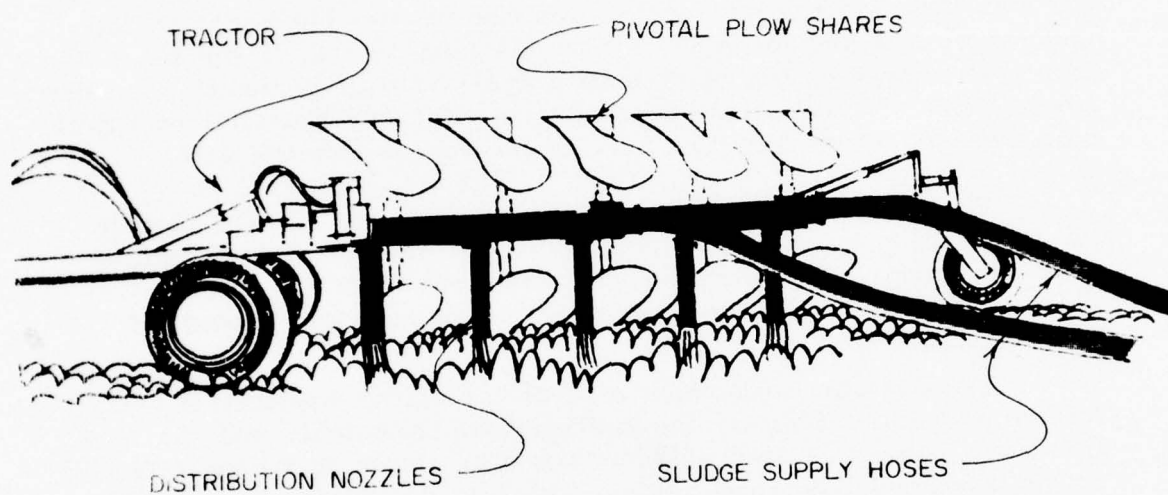
Figure B-IV-C-4

MOLDBOARD PLOW SLUDGE APPLICATOR

ward at a constant rate of speed, lifting the soil up and allowing
sludge to flow freely onto the soil from the nozzle installed behind
it. As soon as the sludge is placed, the lifted soil from the follow-
ing moldboard is deposited over it, covering the injected sludge com-
pletely. In this way potential odor problems are eliminated.

For the physical-chemical sludge, a spray unit as shown in
Figure B-IV-C-5 is connected to a quick coupling by a flexible hose.
A single spray unit serves one pair of laterals extending to both sides
of the header pipe. Each spray unit can complete one cycle of appli-
cation in an eight-month application season per year. The spray unit
moves forward at a constant rate of speed, spraying the sludge onto
the soil from a series of nozzles attached by flexible hoses to the
spray unit. The sludge spray unit is designed so that it can be used
during all stages of the plant growth cycle without producing crop dam-
age. Physical-chemical sludge does not have any potential odor prob-
blems and therefore does not have to be worked into the soil.

The capacity of the pumping station and the size of pipe in
the distribution system is determined by the rate of sludge applica-
tion to be used. The distribution system shown in Figure B-IV-C-3
is a typical system for the application of an advanced biological
sludge.

The specific application area of the sludge application units
for the physical-chemical, the conventional biological, and the land
treatment sludges is also equal to the area shown in Figure B-IV-C-3
for an advanced biological sludge. This is due to the fact that the
capacity of a sludge application unit is limited by the amount of land
that a moldboard plow or sprinkler unit can cover in the time allowed
and not by the type of sludge or by the sludge application rate being
used. The sizes of pumping stations and pipe sizes in the sludge
distribution systems, however, are determined individually because
they will vary according to rate of sludge application required for
each type of sludge involved.

Land reclamation. The utilization of sludge for land reclama-
tion is also designed as a modular system. The land reclamation
area is divided into sludge application units. The sludge, having
been transported to the reclamation area, is first stored in a series
of storage lagoons which can supply sludge to all of the application
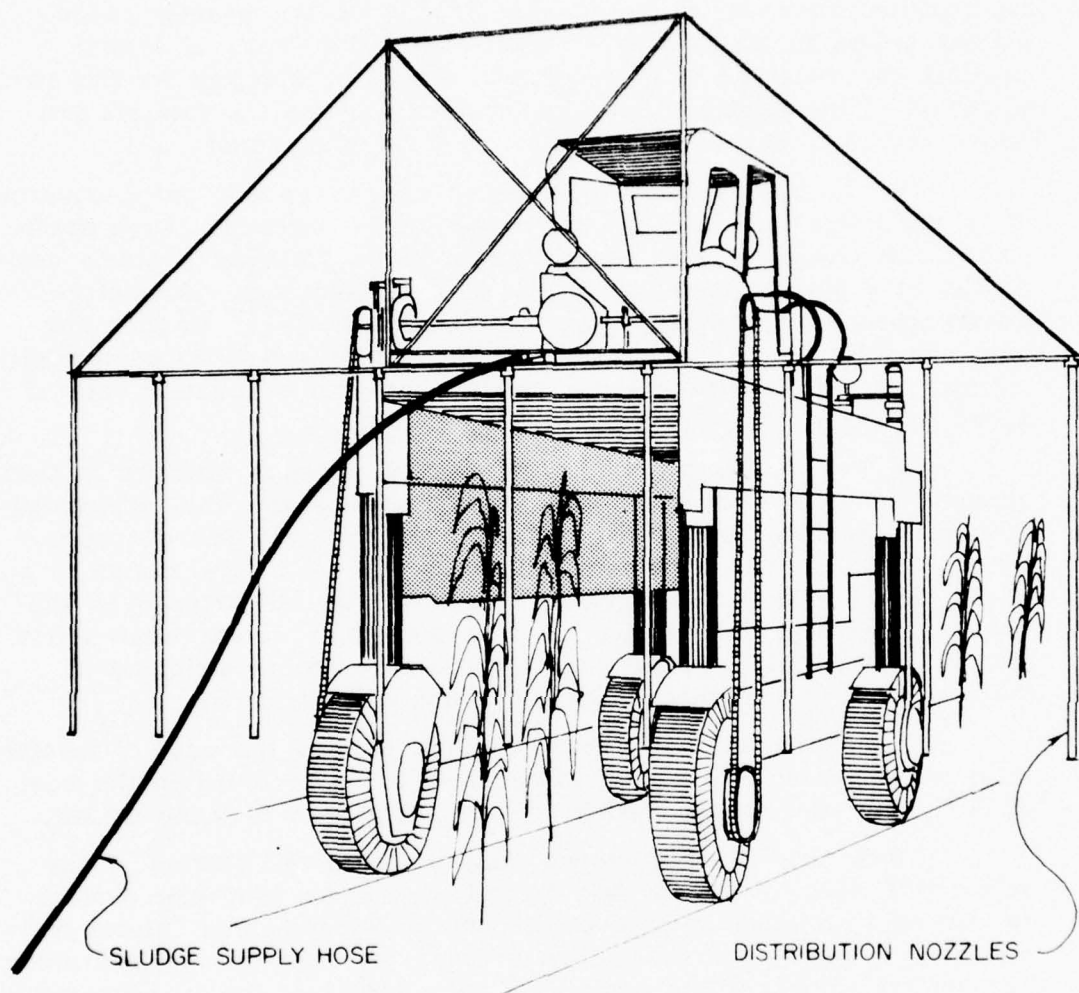units in the area. These lagoons are designed for a ten-day storage
capacity.

SLUDGE SUPPLY HOSE

DISTRIBUTION NOZZLES

Figure B-IV-C-5

PHYSICAL / CHEMICAL SLUDGE APPLICATOR

The application units are positioned in the reclamation area so that one continuous header pipe is installed along the centerline of each application unit. Figure B-IV-C-6 shows the details and a general view of this type of distribution system. This header can be installed above or below ground. There are ten equally spaced access points to the header for each application unit. A lateral pipeline is connected to these access points as required for the application of the sludge. Both the connections and the laterals are above ground to insure ease in operation of this system.

Two lateral pipelines with seven equally-spaced quick-coupling units per lateral are provided for each application unit. Each application unit can also have seven tractor-driven sprinkler systems connected to a single lateral by a pair of flexible hoses. A tractor-driven sprinkler system is shown in Figure B-IV-C-7. In this way only one lateral pipeline can be in operation at a time in each application unit. The tractors and sprinklers move at a constant rate of speed spreading the sludge evenly on the soil.

To provide for the removal of excess ammonia nitrogen in the sludge, if required, a lime feeder is installed at the sludge pumping station. The lime feed rate is adjusted so that the lime concentration in the sludge is sufficient to elevate the pH of the sludge to a point where ammonia dissolved in sludge can be air-stripped as the sludge is sprayed through the air. As previously noted, other forms of nitrogen control could be used instead of ammonia stripping if they proved to be move economical in a given situation.

The system proposed here does not include the cost of leveling strip-mined areas. This cost is assumed to be included in the cost of producing coal as required by recent legislation in many states.

Upon completion of sludge application to the section of the application unit covered by one lateral, the seven sprinkler systems are moved to another section of the unit where the other lateral is installed. The sludge application process can then be resumed using this lateral while, at the same time, the lateral in the just-treated section is moved to the next section to be treated. In this way the entire application unit can be covered with almost continuous use of the seven sprinkler systems.

The capacity of the pumping station and the size of pipe in the distribution system are determined by the rate of sludge application utilized in each particular instance.

LATERAL PIPE

BLINDED WHEN NOT IN USE

VALVE

6' TYP

HEADER (COULD BE ON SURFACE)

DETAIL "A"

2-12" DIA. FLEXIBLE HOSES

QUICK COUPLING

VALVE

LATERAL PIPE

QUICK COUPLING
DETAIL "B"

ONE UNIT

LATERAL PIPES

HEADER

PUMPING STATION

LAGOON

GENERAL VIEW OF APPLICATION SYSTEM

2-12" DIAMETER FLEXIBLE HOSES

QUICK COUPLING SEE DETAIL "B"

SPRINKLER

4620'

SEE DETAIL "A"

LATERAL PIPE

FROM UPSTREAM UNITS

FLOW

TO DOWN STREAM UNITS

HEADER

4620'

660'

UNIT BOUNDARY LINE
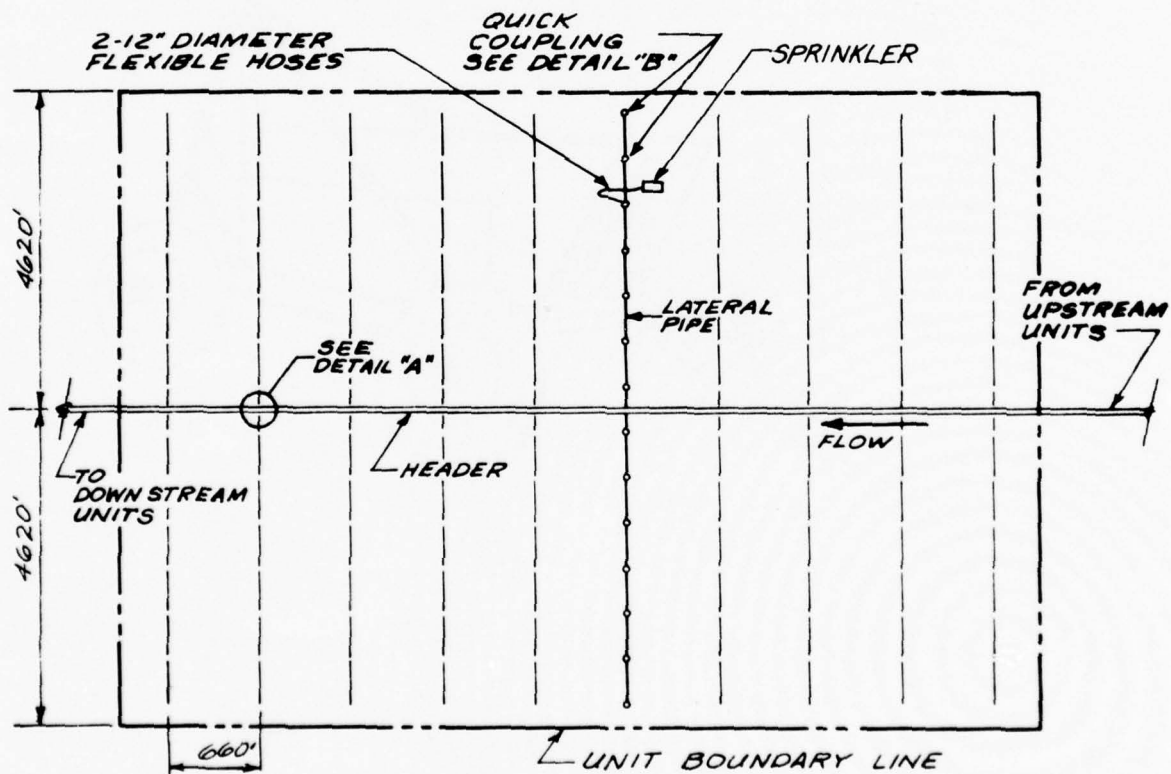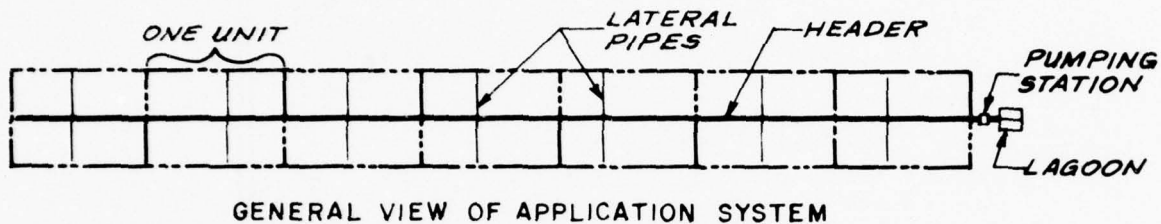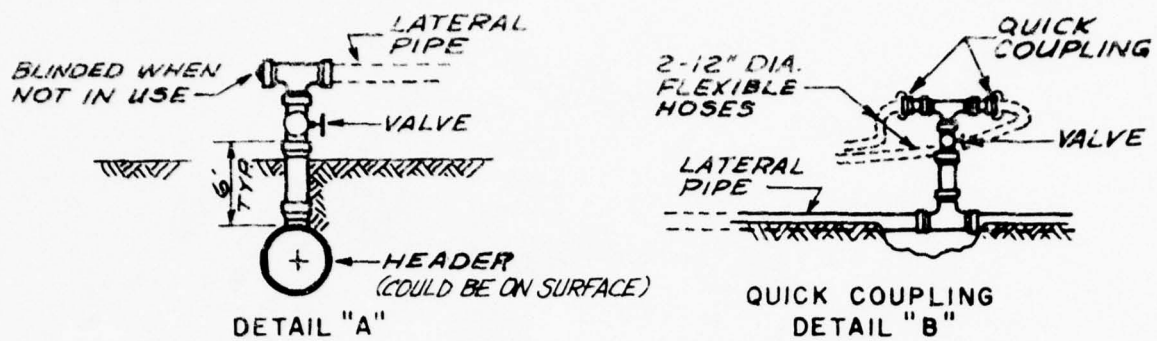
Figure B-IV-C-6

DISTRIBUTION SYSTEM FOR
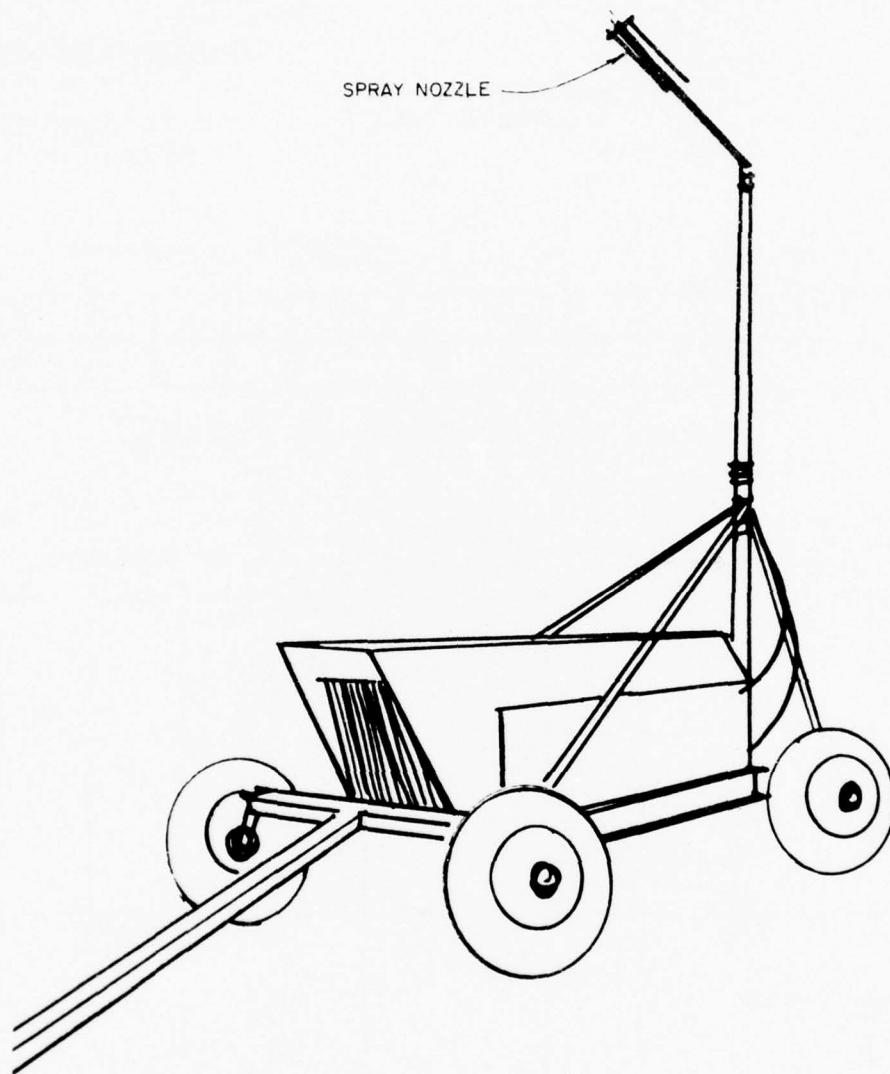SINGLE SLUDGE APPLICATION

SPRAY NOZZLE

Figure B-IV-C-7

SLUDGE SPRINKLER

B-IV-C-23

BIBLIOGRAPHY B-IV-C

COMPONENT BASIS OF DESIGN

1. Ronson AC Communications, Prof. Thomas D. Hinesly, Dept. of Agronomy, University of Illinois, Champaign-Urbana, Illinois.

2. Private communication from Soil Enrichment Materials Corporation, Chicago, Illinois.

D.   STORMWATER MANAGEMENT SYSTEMS

INTRODUCTION

General

Water arrives at the earth's surface as precipitation in the form of rainfall and snow.   Some of the precipitation is returned to the atmosphere in the form of evaporation and transpiration, and some is percolated to deep aquifers so that it does not subsequently appear as runoff.   The remaining portion is identified as stormwater runoff. Stormwater runoff appears in two forms; as infiltrated or groundwater flow, and as overland runoff.

The groundwater portion of stormwater runoff is further divided into two parts.   The first part moves generally horizontally through the soil to nearby lakes and streams and provides their base flows. The second part is intercepted by man-made conduits, such as municipal sewer systems.   This flow is carried to treatment plants, or discharged directly to streams by uncontrolled storm or combined sewers.

The overland flow portion of stormwater is known as surface runoff.   This runoff moves across the surface by force of gravity and usually finds its way into natural drainage such as streams and rivers, or into sewers.

The components of stormwater which are of particular interest to this study are the infiltration into man-made conduits and the overland runoff to streams, conduits, and ditches.   These are the flows which overload our sewer and stream systems and cause flooding problems so familiar to us all.   Flooding occurs because the rate of runoff toward the receiving conduits is greater than the discharge capacity of these conduits.   This is true not only for man-made but also natural waterways.   An important concern which continually increases the flooding hazard, is the burgeoning urbanization of the study area.   As more and more area becomes urbanized, larger surfaces become impervious, causing an increase in flow quantities. In addition, artificial drainage structures accompanying urbanization reduce natural storage and also reduce the time needed for the water to reach our streams.   Both factors cause more serious flooding problems.

Developing metropolitan areas are, therefore, faced with two problems associated with stormwater and directly tied to this urban growth. One is the increased flooding hazards mentioned above. The second is the pollution of the waterways from stormwater which picks up deleterious materials in its passage over the surface and from the overloading of combined stormwater sanitary sewer systems. The obvious solution to this dilemma is to manage the stormwater run-off. The management system outlined in the following sections will provide for orderly, economic and useful handling of these runoff flows. The preferred method of management is the temporary retention of runoff close to its point of origin and its subsequent removal from storage for treatment at a more convenient time, after the threat of flooding and associated pollution has receded.

The C-SELM study considers several alternatives for providing the needed temporary storage. The concept of localized storage was selected as the most viable. There are two reasons for this choice. First, the provision of storage close to points of origin reduces the land area required for any individual storage unit by providing a larger number of such units. Secondly, many existing suburban communities already possess stormwater collection systems which presently discharge into nearby waterways. The connection of these collection systems to localized storage presents far fewer problems and will require far less capital expenditure for new conveyance systems than if storage were to be provided on a larger, regional basis.

The selection of the proper amount of storage is based on the designed effluent quality standards and the expected quality of stormwater spills to streams which occur once the proposed total volume of storage would be filled. Optimum storage is tailored to the land-use of a particular area. It is also based on the best pump-out rate to treatment facilities. Detailed discussion of these aspects of the study follow this introductory section.

Stormwater storage takes different physical forms in different land-use areas, whether urban, suburban or rural.

Urban storage. The urban area of C-SELM is comprised mainly of the 375 square miles of the City of Chicago and several connected suburbs. Storage would be provided for this area in the form of large open pits as a result of a series of studies conducted by local authorities and adopted under the name of Chicago Underflow Plan.[1]

Suburban storage. Suburban areas form a concentric ring around the Chicago metropolitan area and are to be served by the previously mentioned local storage reservoirs. Two types of storage are contemplated: shallow pit storage in areas where open space is still available, and mined storage where land is totally used for suburban development. Other types of suburban storage are provided within areas which presently are rural but which will be suburbanizing by target years of 1990 and 2020.

Rural storage. Areas which are still in the rural category by design year 2020 would be served by local retention basins within each rural watershed. Comprehensive discussion of all aspects of this system is provided in the section on Rural Stormwater Management Systems.

Quantity of Stormwater

The quantity of stormwater which must be managed in the C-SELM area may be estimated with respect to two main factors. The first of these factors is the total acreage of each type of land-use within the area. The second factor is the expected runoff from each type of land use. Land use in the entire C-SELM area is divided into urban, suburban and rural. Urban areas include high-density, residential and heavy industrial sectors. Suburban areas are devoted mainly to residential development, and to a lesser degree, light industrial. Rural land is devoted to cultivation, pasture and farm residential units. Figures B-IV-D-1 and B-IV-D-2, present the areas assigned to each of the three classifications of land use in the C-SELM area for the years 1990 and 2020. Population densities, and manufacturing present in each of the land-use categories are characterized as follows:

1. Urban area. Average population density is 10,000 persons per square mile. The designation of urban land-use was given for population densities of greater than 5,000 persons per square mile and for areas intensively used for manufacturing.

2. Suburban area. Average population density is 4,000 person per square mile. The suburban land-use designation was given for land with population densities between 2,000 and 5,000 persons per square mile with a moderate manufacturing density.

WISCONSIN
ILLINOIS

Harvard

MC HENRY CO.

Marengo

Fox R.

LAKE CO.

WAUKEGAN

ELGIN

KANE CO.

EVANSTON

DU PAGE CO.

Salt Creek

CHICAGO

Lake Mic

AURORA

Fox R.

KENDALL CO.

Sanitary

Ship Canal

Calumet

Sag Channel

EAST CHICAGO

GARY

Little C

R

JOLIET

Du Page R.

Jackson Creek

WILL CO.

Crown Point

Morris

Illinois R.

GRUNDY CO.

ILLINOIS
INDIANA

LAKE CO.

URBAN
SUBURBAN

EVANSTON

CHICAGO

Lake Michigan

MICHIGAN CITY

MICHIGAN
INDIANA

EAST CHICAGO

GARY

La Porte

Little C. R

LA PORTE CO.

Crown Point

PORTER CO.

LAKE CO.

ILLINOIS
INDIANA

Kankakee R

MILES
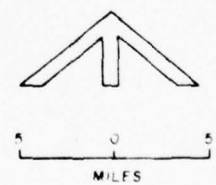
Figure B-IV-D-1
1990 LAND USE

B-IV-D-4

URBAN

SUBURBAN

EVANSTON

CHICAGO

Lake Michigan

MICHIGAN CITY

MICHIGAN
INDIANA

EAST CHICAGO

GARY

La Porte

Little Calumet *R.*

LA PORTE CO.

Crown Point

PORTER CO.

Kankakee R.

LAKE CO.

ILLINOIS
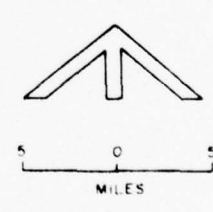INDIANA
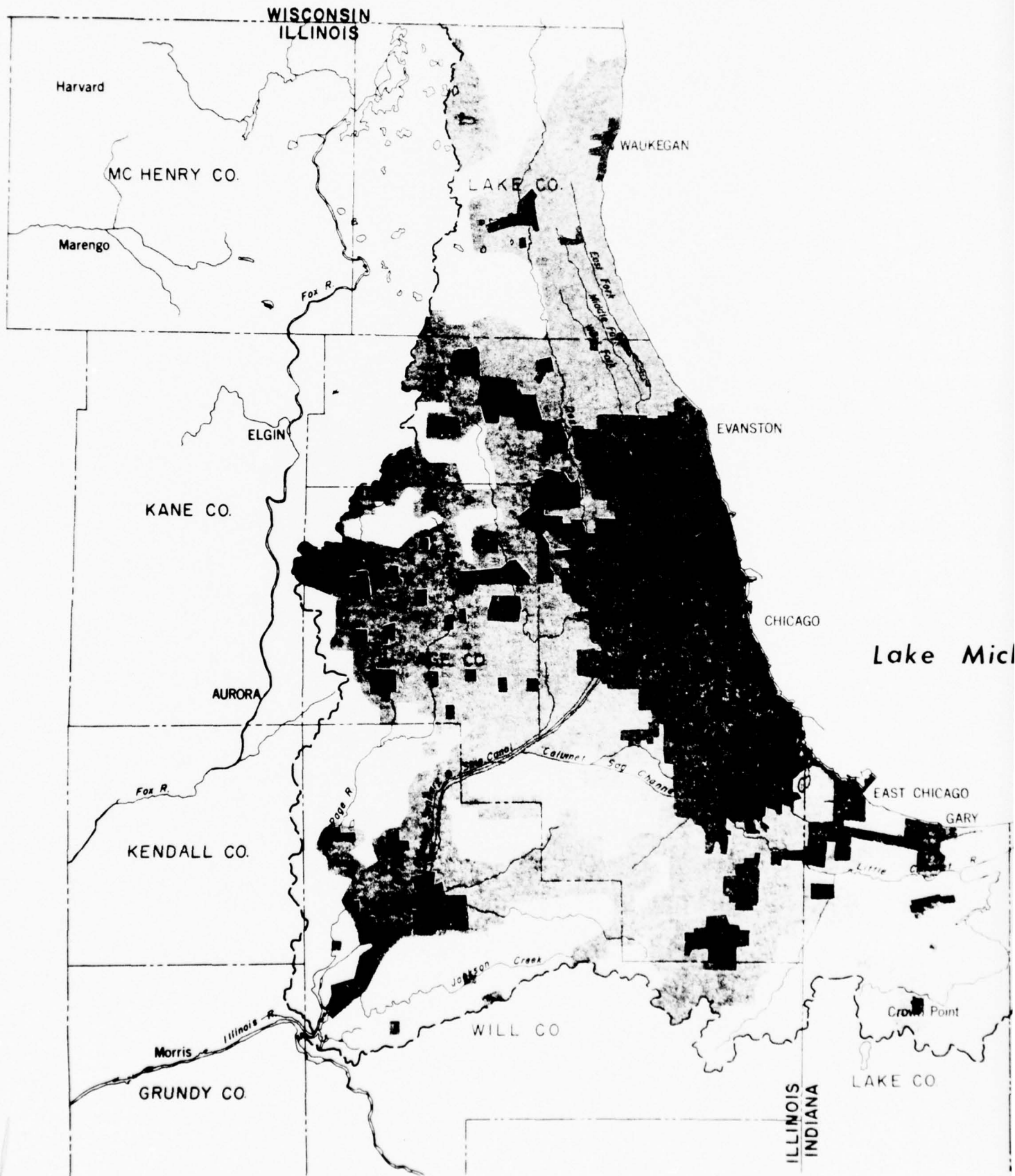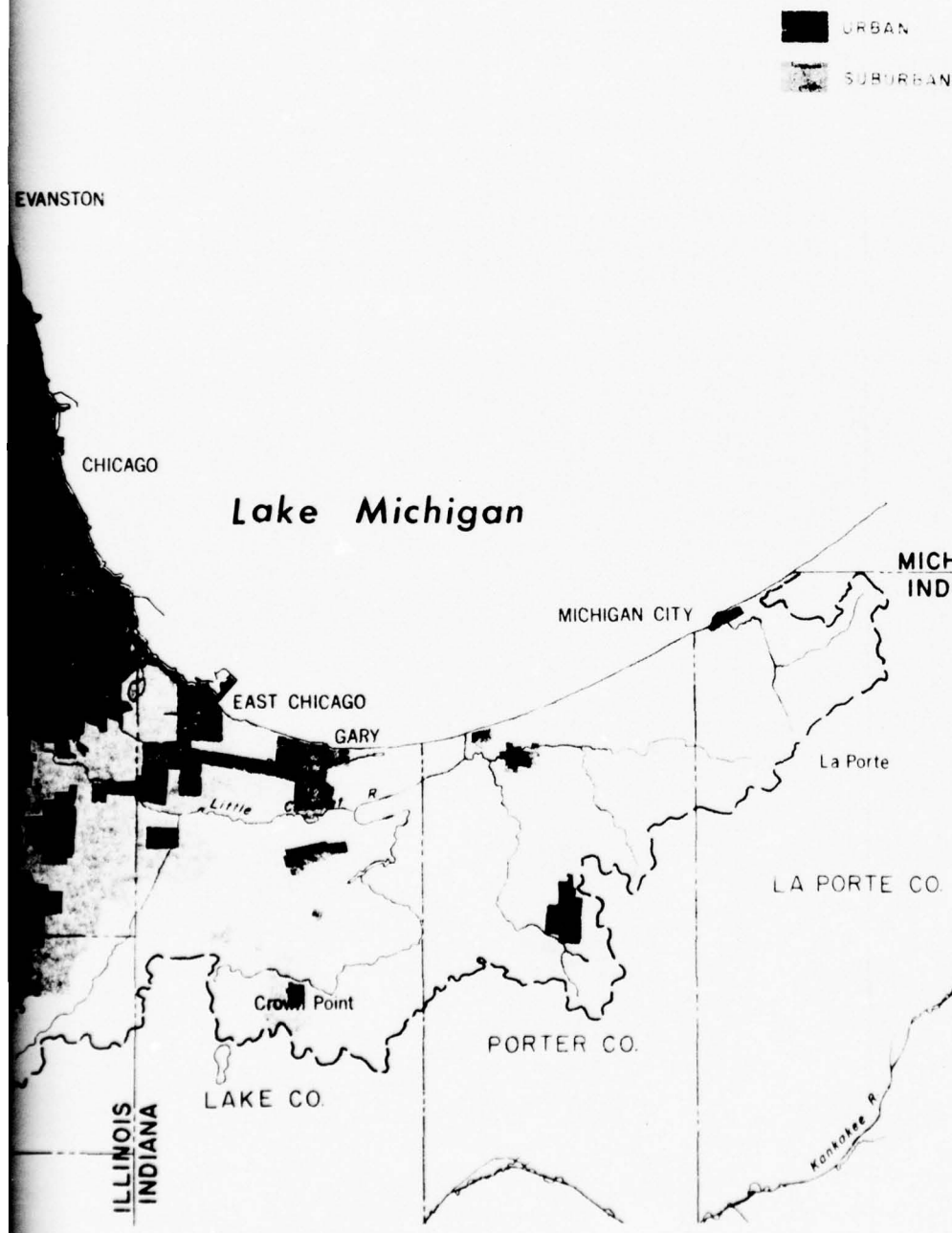
Figure B-IV-D-2
2020 LAND USE

B-IV-D-5

5 0 5
MILES

3. Rural area. Average population density is 1,000 persons per square mile. Rural area land-use included all population densities fewer than 2,000 persons per square mile.

Further application of these land use categorizations is made in the Appendix B, Section III.

Generalized relationships between type of land-use and the amount of stormwater runoff can be developed. Based on flow data derived for the Chicago combined-sewer area for a period of 21 years, and stormwater runoff projections, it is estimated that about 19 inches of runoff per year may be expected from underline urban areas.

Studies of C-SELM rural area runoff indicate that 10 inches of runoff per year may be expected. The study of average discharge (in inches per year) from Hickory Creek, Long Run Creek, DuPage River, Deep River, and Burns Ditch covered periods of record varying in length from 19-30 years.

Suburban areas, with their land use characteristics closer to rural than urban land use, were projected to contribute 12 inches of runoff per acre per year. The following tabulation shows the results of studies conducted on stormwater runoff in the C-SELM area:

Land Use vs. Average Annual Runoff

Land Use	Average Annual Runoff in Inches/Acre
Urban	19
Suburban	12
Rural	10

Stormwater flows expected in the C-SELM area for design years 1990 and 2020 are presented in Table B-III-C-2 in Appendix B, Section III-C.

Actual quantities of stormwater during any year may vary markedly from the average values enumerated above. This may, for example, be due to a prolonged wet period preceding a given storm. Soils during such wet periods become saturated and do not accept

additional infiltration of rainfall. Similar situations occur during the winter when the soil is frozen. The sudden thawing of snow and ice could produce flash floods. Topography of a watershed also has a marked influence on the amount of runoff expected. Steep gradients are less conducive to infiltration than flat areas; consequently, larger amounts of rainfall become surface runoff. All of these conditions typically produce higher-than-average ratios of runoff volume to precipitation volume.

The surface runoff originating within a given watershed eventually finds its way into a stream via storm drains or overland flow. The highest rates of discharge ordinarily prevail at any point in the watershed at the time when the effect of the storm runoff from the entire tributary area has reached this point. This time is dependent upon the time required for the effect of runoff from the farthest portion of the watershed to reach that point, called the time of concentration. Since average rainfall rates decrease with duration of rainfall, the shorter the time of concentration, the higher the design rate of discharge per unit area of a given probability of occurrence. The time is shortest for small, broad, steep areas with rapidly shedding surfaces. It is increased by dry soil, surface detention, vegetal cover, and storage.

In consideration of all the many variable factors involved which affect the amounts and rates of runoff to be expected from a given area, it is clear that the best projection of runoff can only be derived from direct measurements obtained in the streams and other stormwater conductors. Studies have been conducted for urban and suburban-rural runoff quantities.

The urban documentation is based on the results of a study [1] conducted by the City of Chicago in the "Runoff Simulation Model" [1] for the 375 square mile area of Chicago and the several high-density suburbs served by the same combined sewer system. This area, taken as a unit, is characterized as completely urbanized. The study uses a 21-year period of record, from 1949 through 1969, during which historical hourly rainfalls, at each of more than 20 rainfall gaging stations, are taken into account. The end result of this study is a statistical relationship between number of storm events and storage required to contain any given runoff and the number of times, during the 21-year period, in which the amount of storage proposed is insufficient to contain the historical runoff. Storage is defined in terms

of inches of runoff from a given contributing watershed. When reference is made to 2.5 inches of storage, for instance, this can be translated into a volume of storage by multiplying by the respective area. For example, the area served by combined sewers in the MSD area is 375-square miles or 240,000 acres. In terms of acre-feet of storage, 2.5 inches of runoff storage converts to 50,000 acre-feet. The relationship for the 375-square mile area is shown in Figure B-IV-D-3.

The suburban-rural runoff data is presented in this study, using records of flows from four watersheds in the C-SELM area. The elected watersheds are Salt Creek (Illinois), Thorn Creek, Des Plaines River and Hickory Creek. They represent watersheds of various drainage patterns, ground covers, and degrees of development. Approximately 130 storms were analyzed, based on historical flow records for the same 21-year period analyzed in the urban study. The runoff for a particular storm was calculated by subtracting the base flow from the accumulated total flows throughout the storm period.

The amount of runoff for each of the 130 storms, expressed in terms of inches of runoff over the drainage area, was tabulated in descending order. As in the urban runoff study, the suburban-rural runoff contemplated the use of storage to prevent spillage of untreated stormwater into the waterways. The required storage volumes are plotted graphically in Figure B-IV-D-4 against the number of events of overflow. It can be noted here, for future reference, that despite the appreciable differences in stream characteristics and watershed development, the four watershed curves as well as the urban curve show distinct similarities, particularly the definite discontinuity of curves in the vicinity of 2.5 inches of storage volume. The corresponding number of overflow events in 21-years which would not be contained by 2.5 inches of storage ranges from 2 to 4 events.

Quality of Stormwater

Stormwater, contrary to the belief of the man on the street, is not a clean and uncontaminated media in which one washes one's hair. In many instances, the stormwater contaminant load is just as great as that of municipal and industrial flows. Therefore,
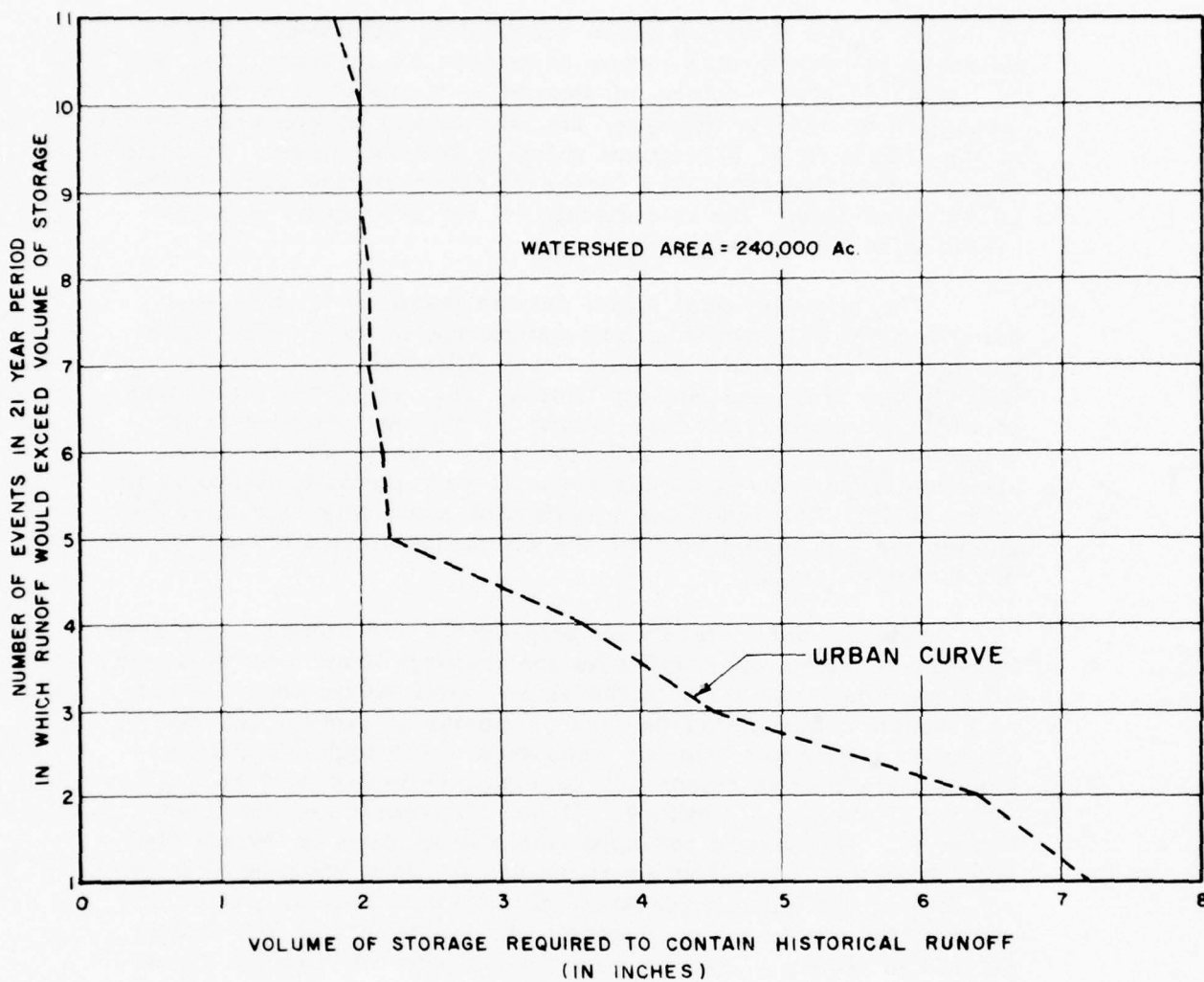
Figure B-IV-D-3
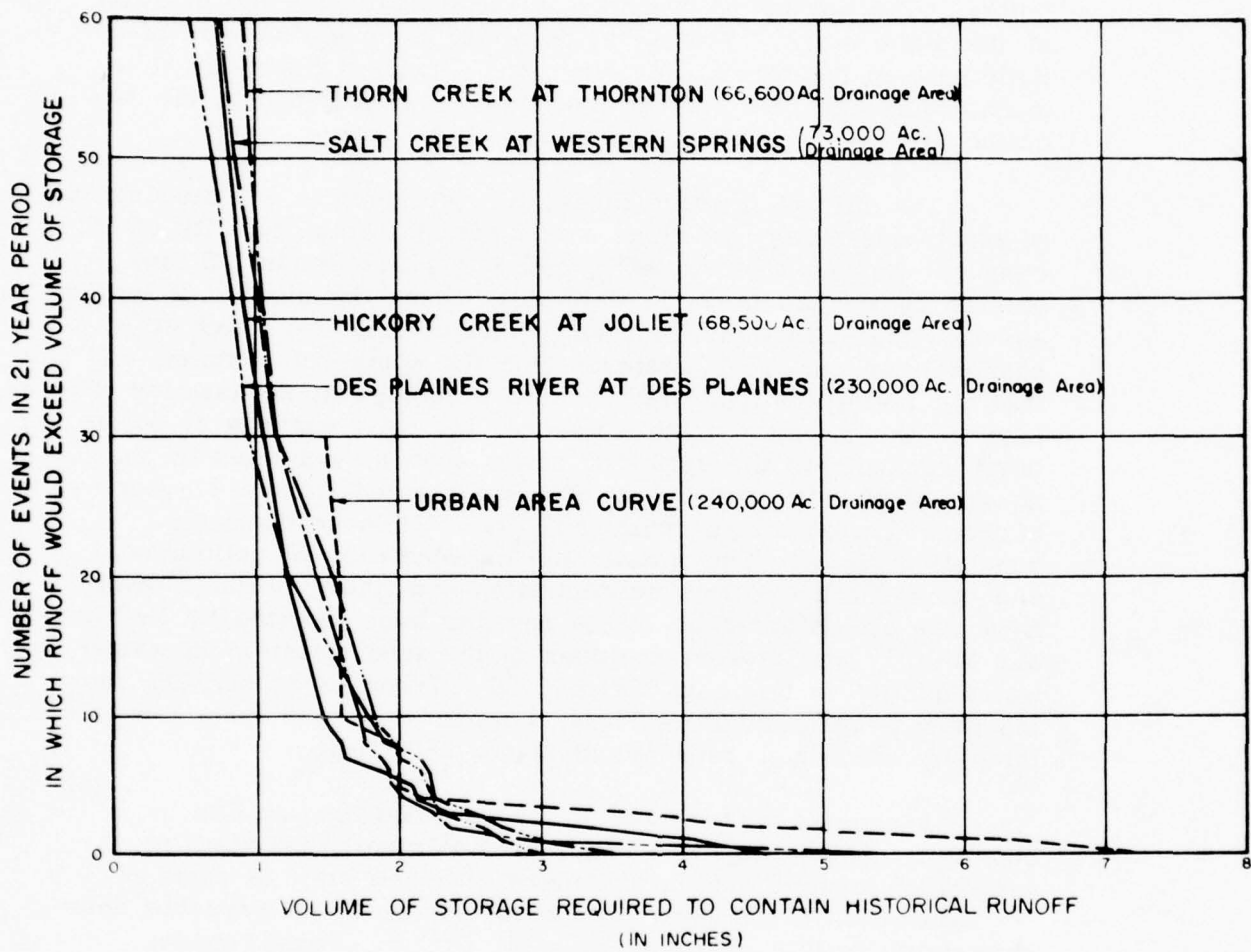
STORMWATER STORAGE VOLUMES
URBAN AREA

B-IV-D-9

Figure B-ⅣV-D-4

STORMWATER STORAGE VOLUMES
URBAN, SUBURBAN AND RURAL AREAS

it is of great importance to treat stormwater contamination in a fashion similar to other wastewater contamination, and to realize its impact on the total water quality goals expressed in the NDCP policy. The quality of overflows is dependent upon the duration of the storm event. For short durations, the concentration of pollutants in the runoff is rather high. In some instances it is *much higher than are the corresponding* concentrations in the so-called dry-weather municipal and industrial flow.

The City of Chicago Study, as mentioned in the discussion of runoff quantities, develops a concept of a total *quantity of Bio-*chemical Oxygen Demand (BOD) and Suspended Solids (SS) available to be washed from the watershed during the storm. It further assumes that this total quantity is washed into the sewer at successively diminishing rates during the course of a storm, and that the pollutants regenerate on the watershed at an assumed rate between the storms. In this manner, the total quantity of pollutants washed into the combined sewer system, indicated by the number of pounds of BOD and SS, is estimated for the 21-year historical period studies with the City of Chicago computer simulation model. The initial high concentration of pollutants, and the reduction of this concentration with the duration of over-flow was also studied by others and has been reported by DeFilippi and Shih,[2] who present analyses of the time variation in waste-water quality in terms of BOD and SS. Examples of curves, showing these parameters for a short, intense storm and for a low-intensity storm are presented in Figure B-IV-D-5.

A general tapering off of the pollutant loading can be observed in both instances and indicates clearly the need of capture of initial flushing of sewers after the start of overflow. For the suburban and rural watersheds, there is no available data comparable to that available from the City of Chicago model. However, studies conducted by DeFilippi, et. al., characterize the water quality of separate-sewer storm-runoff as poor, with organic and nutrient concentrations to be approximately one-third of those in combined sewer discharges. In all studies, the total SS concentrations are reported to be higher in storm sewers than in combined sewers.

The most feasible way of preventing large quantities of pollutants from spilling into the waterways of the C-SELM area is

JUNE 8, 1969 (5:50 PM) 13 MIN. —0.7"

SHORT, INTENSE STORM IN COMBINED SEWER

LOW INTENSITY STORM IN COMBINED SEWER
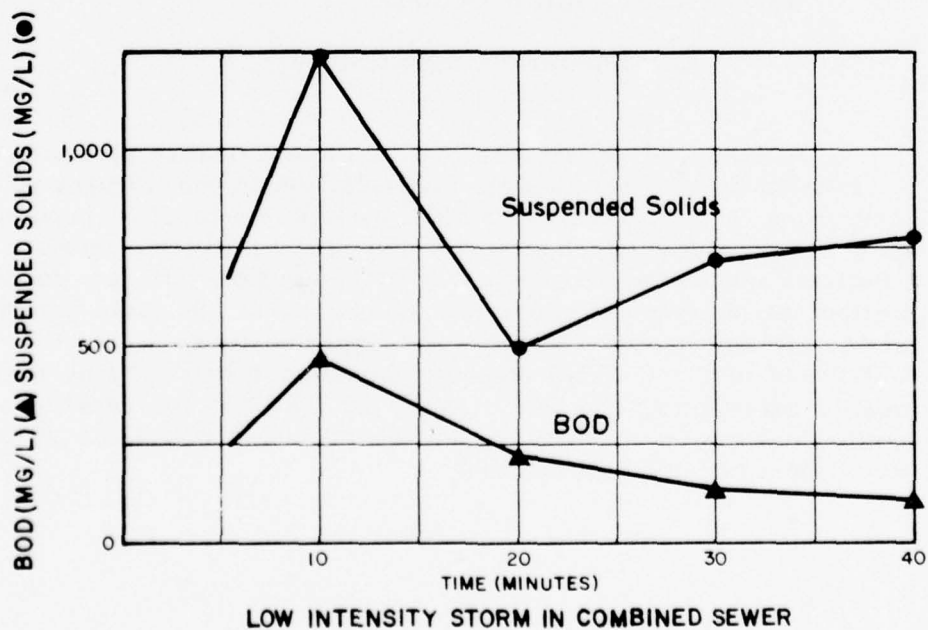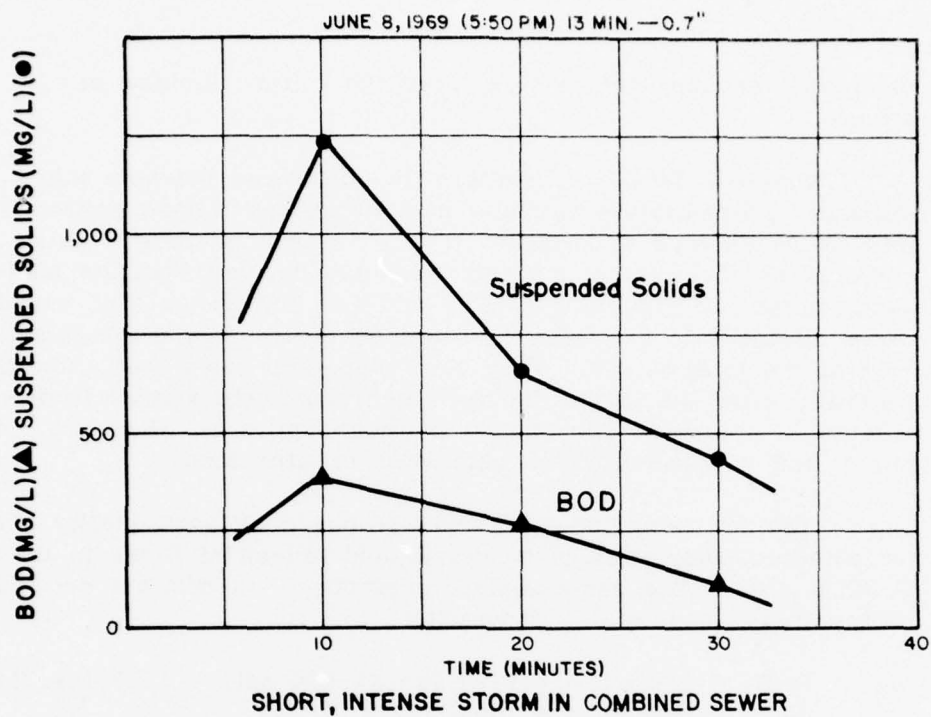
Figure B-Ⅳ-D-5

WASTEWATER QUALITY

B-IV-D-12

to provide enough storage to capture the initial flushing of sewers.

Figure B-IV-D-6 illustrates the difference between the amounts of floodwaters captured and stored, and those spilled into the waterways at some arbitrarily assumed storage. This curve is an idealized runoff-storage curve derived from the analogous curves in Figures B-IV-D-3 and 4. The amounts of untreated water spilled into the waterways are obviously only a small percent of the total runoff. They also represent flows that enter the waterway after the initial flush of highly polluted waters is deposited in storage. The quality of these spills is, therefore, much higher and contamination of waterways is minimized.

The following table presents proposed effluent quality goals for selected parameters of municipal and industrial flows in the C-SELM area. The same goals are proposed for effluent derived from captured and treated stormwater.

$$BOD_5 = 2 \text{ mg/l} = 0.0167 \text{ lbs./1,000 gal.} = 16.7 \text{ lbs./MG}$$
$$SS = 0 \text{ mg/l} = 0 \text{ lbs./MG}$$
$$P = 0.01 \text{ mg/l}^a = 0.0000834 \text{ lbs./1,000 gal.} = 0.0834 \text{ lbs./MG}$$

Where, MG = Million Gallons

[a]Land Treatment System Only

Inclusion of phosphorus in the effluent quality parameters is dictated by the fact that its concentration in the effluent is a controlling factor for algal growth. A reduction of algal populations to a background level--that which can still be found in some relatively unpolluted lakes and streams--requires very low concentrations of phosphorus, approaching 0.01 mg/l. Selected parameters of the effluent quality of stormwater expected to spill into the waterways of the C-SELM area are tabulated below. Using these quality parameters, one may multiply by the expected quantities of spilled stormwater to calculate the quantities of pollutants which would be associated with them.
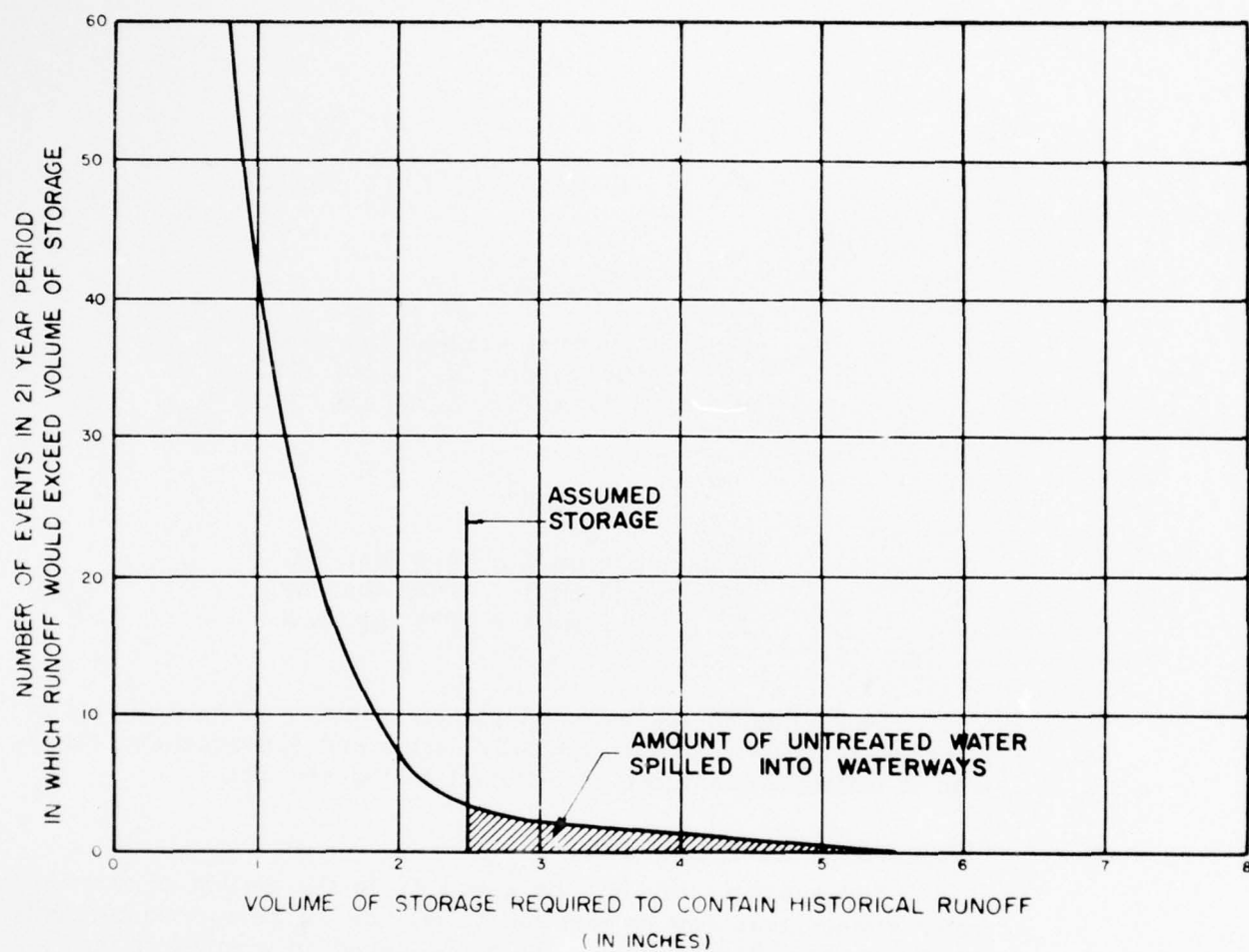
Figure B-IV-D-6

GENERALIZED STORMWATER STORAGE VOLUMES

STORMWATER QUALITY PARAMETERS
PRIOR TO CAPTURE AND TREATMENT

Urban[2]/

BOD = 10 mg/l = 83.4 lbs./MG
SS = 130 mg/l = 1,084 lbs./MG
P = 1 mg/l = 8.34 lbs./MG

Suburban[3]/

BOD = 20 mg/l = 166.8 lbs./MG
SS = 500 mg./l = 4,170 lbs./MG
P = 0.25 mg/l = 2.085 lbs./MG

Rural[4]/

BOD = 10 mg/l = 83.4 lbs./MG
SS = 550 mg/l = 4,587 lbs./MG
P = 1 mg/l = 8.34 lbs./MG

Depending on the selected storage volume, the quantity of stormwater spilled into the waterway varies and subsequently carries with it different quantities of pollutant to the streams.

Effluent Quality vs. Storage

The question may now be asked as to the amount of stormwater storage that may be required to provide for postulated effluent quality goals. A second question to be asked is whether or not the pollutant load from spillage of untreated stormwater is sensitive to small variations in the proposed storage quantities.

In order to answer these questions, a calculation was made of the amounts of Biochemical Oxygen Demand (BOD), Suspended Solids (SS), and Phosphorus (P) in the spilled untreated stormwater. A comparison of these amounts with the amounts of the pollutants contained in the effluent of treated municipal and industrial flows

is important. If the pollutant loading from the spilled untreated stormwater exceeds the normal expected loadings of municipal and industrial, then the proposed quality goals cannot be met. Consequently, one might expect that an increase in storage would create a beneficial effect by reducing the number of spills, the volume of individual spills and the total pollutant load.

For a basis of comparison, the pollutant loads in municipal and industrial flows, as well as in the spilled stormwater flows, are tabulated and presented in Table B-IV-D-1 both in pounds per acre per event of spill, and as average pounds per acre per year. Due to the long-range effects of phosphorus and suspended solids on the quality of water, these two parameters can best be viewed on the average yearly basis as tabulated in Table B-IV-D-1. Biochemical oxygen demand, on the other hand, has an immediate, short range, effect on water quality, and should be considered on the event-of-spill basis.

Conclusions

The conclusions that can be drawn from Table B-IV-D-1 insofar as suspended solids loadings are concerned are that, since the allowable loading is approximately zero, no amount of provided storage can satisfy this parameter except storage of the total amount of runoff. For economic reasons, providing for capture of all the runoff arriving at the C-SELM area is impractical. Therefore, the suspended solids loadings contained in spills from the provided storage must be accepted as a practical reality. Every possible management practice should be employed to minimize the suspended solids concentration associated with these spills.

A distinct discontinuity exists in the storage vs. events of spill curves shown in Figures B-IV-D-3 and D-4. This discontinuity occurs at about 2.5" of storage. With reference to the phosphorus loadings in spills as presented in Table B-IV-D-1, storage amounts larger than 2.5" are beneficial in further reducing the number of spills (from four in 21 years). The amount of benefit associated with any provision for significantly more storage is offset, however, by the cost connected with such a provision. The incremental benefit obtained by providing additional storage in excess of 2.5" does not justify the incremental cost.

B-IV-D-16

TABLE B-IV-D-1

POLLUTANT CONTENT IN SPILLS AT DIFFERENT STORAGE QUANTITIES

Pollutant Content in Effluent

Area	Storage Provided	BOD #/ac./event		BOD #/ac./yr.		SS #/ac./event		SS #/ac./yr.		P #/ac./event		P #/ac./yr.	
		Allow.	Act.	Allow.	Act.	Allow.	Act.	Allow.	Act.	Allow.	Act.	Allow.	Act.
Urban	1.5"	.182	2.42	18.7	2.08	0	31.4	0	27.1	.0009	.242	.0934	.208
	2.5"	.409	5.55	18.7	1.155	0	72.12	0	15.01	.00205	.555	.0934	.115
	4.4"	.205	2.69	18.7	0.385	0	35.03	0	5.00	.0010	.269	.0934	.0385
Suburban	1.0"	.409	6.57	18.7	5.66	0	164.2	0	141.5	.00205	.082	.0934	.0708
	2.85"	.272	4.36	18.7	1.246	0	109.05	0	31.14	.00136	.055	.0934	.016
Rural	1.5"	.626	2.94	18.7	1.042	0	161.9	0	57.3	.00313	.294	.0934	.104
	2.5"	.439	2.06	18.7	.412	0	113.3	0	22.67	.00219	.206	.0934	.0412
	3.4"	.215	1.02	18.7	.097	0	56.05	0	5.34	.00107	.102	.0934	.0097

Effects of BOD on the quality of water immediately after the occurrence of a spill are also of interest to this discussion. The background effluent quality goal for BOD = 2 mg/l. The average expected value of BOD in a spill is calculated to be 10-12 mg/l. An examination of Table B-IV-D-1 demonstrates that no reasonable amount of storage can be expected to satisfy the single event inequity between allowable and actual BOD loading. The resulting excess oxygen demand in the C-SELM waterways subsequent to a stormwater spill can be satisfied by a combination of the following mechanisms: utilization of in-situ dissolved oxygen, reaeration due to natural flow and supplemental downstream induced aeration.

A system of detection devices might be provided throughout the C-SELM waterways to record the BOD requirements on a stream-by-stream basis. Additional aeration facilities, possibly in the form of in-stream bubbler systems or riffle dams can be provided at strategic locations along the streams and waterways for the purpose of injecting additional oxygen to satisfy the aerobic requirements of a healthy aquatic environment. These considerations require detailed studies on a stream basis and are mentioned here only to indicate feasibility. The spills discussed here occur on the average of four times in 21 years of record. The regenerating characteristics of streams must be taken into consideration in a detailed analysis before any recommendation is made for additional aeration facilities to be provided in any particular location along the waterways of the C-SELM area.

In summary, control of suspended solids is not economically justified beyond that associated with approximately 2.5" of storage. A storage volume of approximately 2.5" will satisfy the reasonable quality goals associated with phosphorus. The biochemical oxygen demand loading contributed by a stormwater spill can be mitigated by natural and possibly artificial instream reaeration at critical locations.

SELECTION OF STORAGE

Storage Parameters

The design parameters for storage selection include land use classifications, desired water quality, pump-out rate and cost.

Urban-Suburban Storage Selection

Urban pump-out rates. The amount of storage required decreases as the pump-out rate increases for any given volume of storm runoff. The effect of the pump-out rate on the volume of storage required is much more significant for larger storms which usually have long duration. The relationship between these two parameters can be identified by a mass-curve analysis. In this report, a computer study conducted by the Bureau of Engineering, City of Chicago for the 375-square mile combined sewer area is used to derive the urban curve.[1] To obtain the suburban-rural curve, the continuous 21-year stream records of four major watersheds were used to obtain the mass-curve. Streams analyzed were the Des Plaines River at Des Plaines, the North Branch of the Chicago River at Niles, Salt Creek at Western Springs, and Thorn Creek at Thornton. The pump-out rates tested in the computer analysis were 500 cfs, 1,000 cfs, and 1,500 cfs. Dividing these flow rates by the urban contributing area of 375-square mile = 240,000 ac. one obtains the following:

$$500 \text{ cfs} = 0.00208 \text{ cfs/ac.} = .00208 \text{ inches/hr.}$$
$$1,000 \text{ cfs} = 0.00416 \text{ cfs/ac.} = .00416 \text{ inches/hr.}$$
$$1,500 \text{ cfs} = 0.00624 \text{ cfs/ac.} = .00624 \text{ inches/hr.}$$

Suburban pump-out rates. Suburban pump-out rates selected for the analysis are .001 cfs/ac., .002 cfs/ac., and .004 cfs/ac. The mass curves for each year of record for each suburban-rural stream, previously mentioned, are plotted with total flows shown in cubic feet per second per day. Also shown on the diagrams are base flows and the three pump-out rates to be tested. The maximum difference between a particular pump-out line and the mass-curve constitutes the maximum daily flow volume which must be stored in order to prevent overflow in excess of the designated pump-out rate.

Figure B-IV-D-7 shows a typical mass curve diagram for the Des Plaines River for the year 1965. The values of storage required are converted from cfs/day to inches of storage. For example:

$$43,000 \text{ cfs/day} = \frac{43,000 \times 60 \times 60 \times 24}{43,560} = 85,290 \text{ ac-ft}$$

$$85,290 \text{ ac-ft} /359 \times 640 = 0.371 \text{ ft.} \times 12 = 4.46''$$

The values are from Figure B-IV-D-7 shown below and represent the maximum storage required for a pump-out rate of .001 cfs/Ac and the Des Plaines watershed area of 359 square miles. These values

DATE — 1965
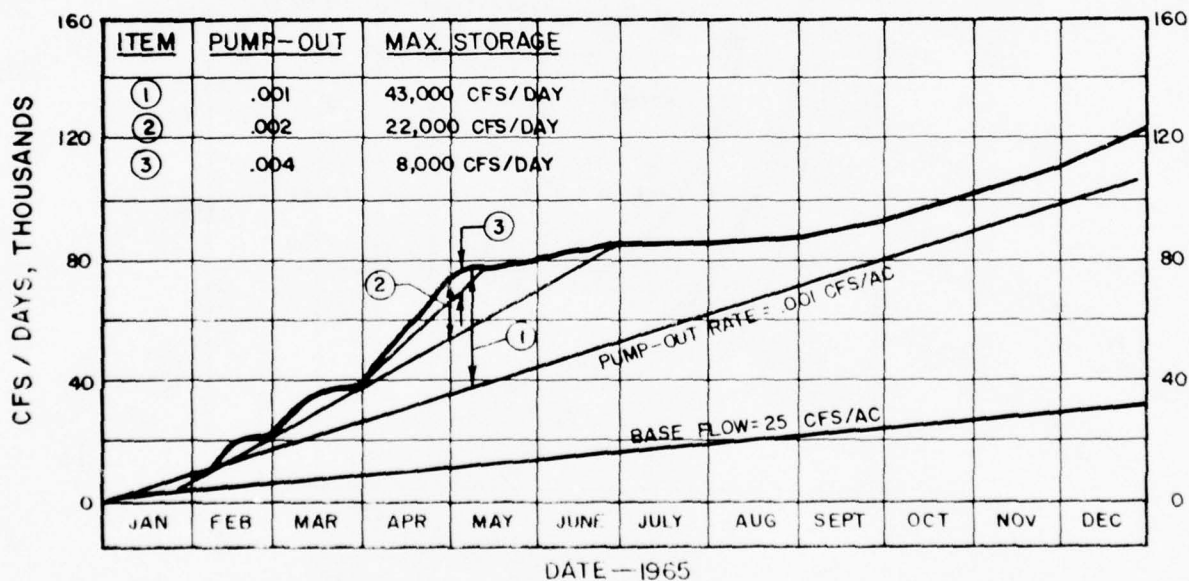
Figure B-IV-D-7

MASS CURVE FOR DESPLAINES RIVER

were tabulated and used for the selection of the design storm. The
fifth largest storm, as defined by total volume of overflows or the total
total volume of mass runoff was selected for the cost optimization
analysis. This fifth storm was selected from among the tabulated
values and is plotted on Figure B-IV-D-8 for the four suburban curves
as well as the urban curve and the generalized suburban curve.
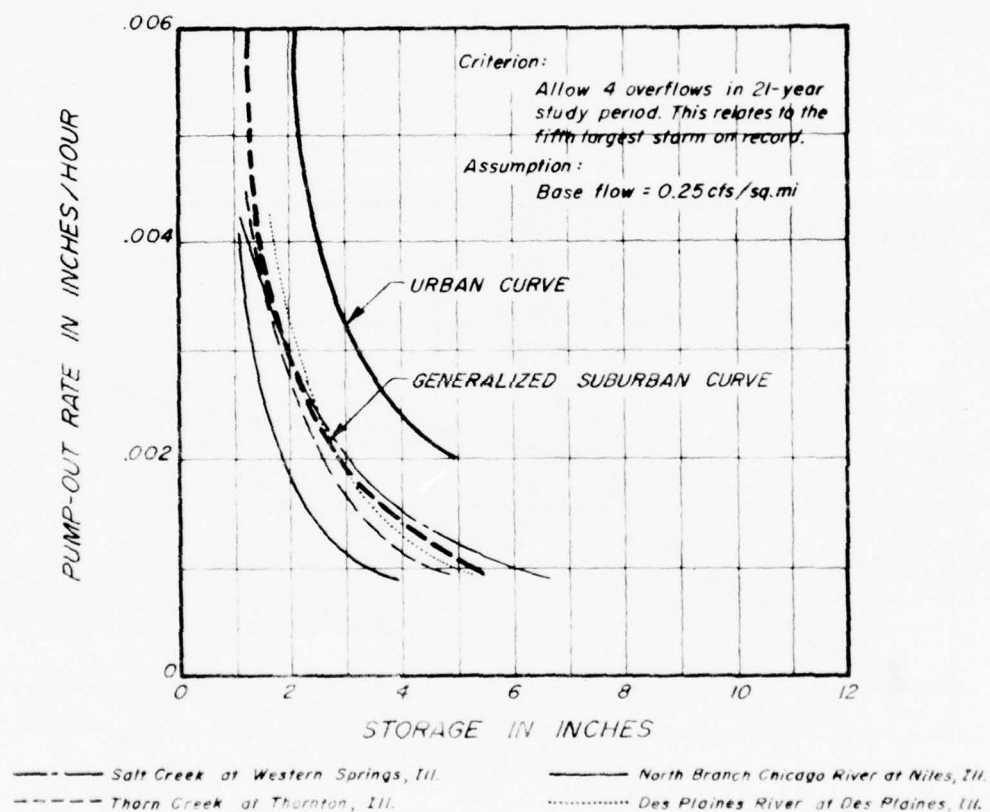
Figure B-Ⅳ-D-8

PUMP—OUT RATE vs STORAGE REQUIREMENT

Optimization. In the cost optimization analysis, three major cost parameters are considered. They are cost of storage, cost of conveyance systems, and cost of treatment. Cost optimization for three types of treatment under consideration are performed for urban and suburban areas.

Urban

Cost optimization for the urban area is based on costs proposed in the Chicago Underflow Plan. Additive costs of storage and conveyance to treatment are plotted for each pump-out rate for the three technologies of treatment. Figure B-IV-D-9 shows the resulting optimization curves.

From Figure B-IV-D-9 the cost is optimized for all three treatment technologies at pump-out rates varying from 0.003 cfs/ac. to 0.004 cfs/ac. For the purpose of this study, a pump-out rate of 0.004 cfs/ac. is selected for the urban areas. From Figure B-IV-D-8 the corresponding value of storage is then selected to be 2.5". A more detailed development of cost optimization for the Chicago Underflow Plan may dictate further refinements in the amount of storage and pump-out rates. But, for the purposes of this study, the pump-out rates and optimized storage values given above are adopted.

Suburban

Optimization of the suburban costs versus pump-out rates was performed in similar manner to the urban optimization. A typical drainage basin of 25 square miles was selected and optimization of cost of storage, conveyance, and treatment systems was performed on this basis. Corresponding suburban cost optimization curves for the three treatment technologies are shown on Figure B-IV-D-10.

The corresponding optimized pump-out rate varies from 0.002 cfs/ac. to 0.003 cfs/ac. A selection of 0.002 cfs/ac. is made for the purpose of this study. From Figure B-IV-D-8 a storage of 2.85 inches can be selected to correspond to the above pump-out rate.
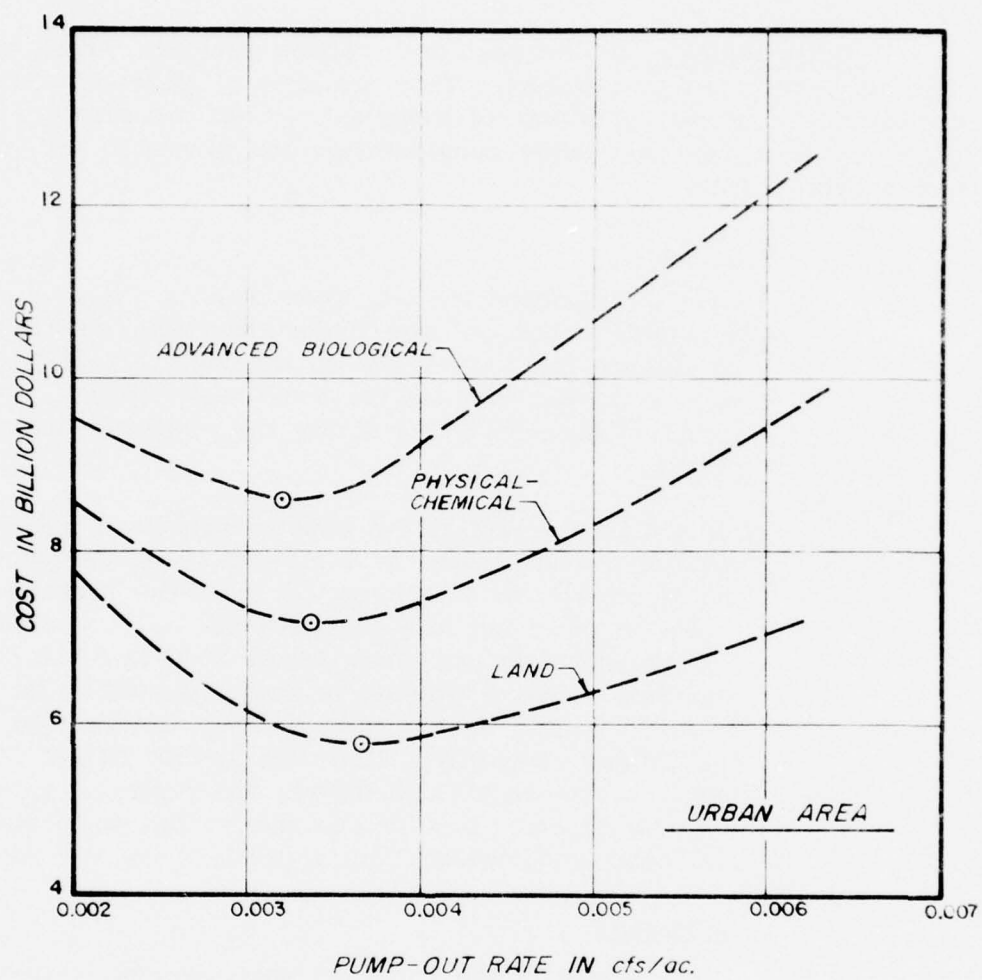
Figure B-IV-D-9
PUMP-OUT RATE OPTIMIZATION CURVES
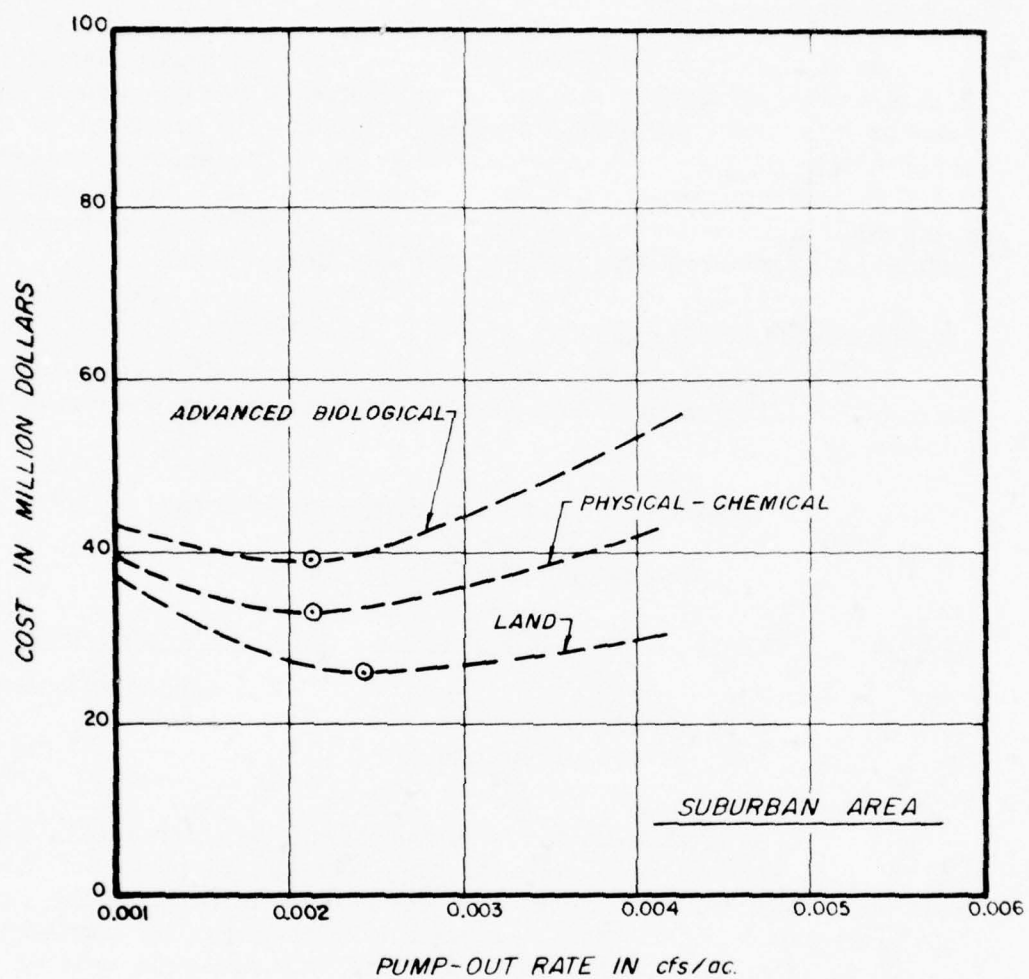
Figure B-IV-D-10

PUMP-OUT RATE OPTIMIZATION CURVES

Rural Storage Selection

An annual rural runoff of 10 inches was previously established for this study. Suburban storage is estimated to be 12 inches. Because of this small variation, a rural pump-out rate identical to the suburban rate of 0.002 cfs/ac. was selected. The storage amount of 2.5 inches was based on a realization that rural areas managed by the rural stormwater management systems afford increased opportunities for stormwater to infiltrate into the groundwater.

Summary of Storage Selection

The values of storage and pump-out rates that are used in designing C-SELM storage and conveyance systems for stormwater are as follows:

Values of Storage and Pump-Out Selected
for Design of C-SELM
Storage and Conveyance Systems

Land Use	Pump-Out Rate .002 cfs/ac.	Pump-Out Rate .004 cfs/ac.
Urban	-----	2.5 inches of storage
Suburban	2.85 inches of storage	-----
Rural	2.5 inches of storage	-----

As the rural area begins to suburbanize, an increase in storage need of 0.35 inches, or a corresponding increase in pump-out rate might be anticipated. However, the techniques utilized in the rural stormwater management (land management system) can be carried forth into its suburban use mode. The result of this approach will be reflected in the retention of more runoff in the remaining open areas and the consequent continuation of a storage requirement of 2.5" as in the rural mode will be appropriate.

STORAGE SYSTEMS

Storage Requirements

How are the requirements for storage of stormwater to be met? How can these large quantities of storage be provided through the C-SELM area? Total required quantities of acre-feet of storage for urban, suburban and rural land use are:

Land Use	Ac.-Ft. Storage, 1990	Ac.-Ft. Storage, 2020
Urban	63,533	75,400
Suburban	119,138	163,324
Rural	178,627	128,000

As the land use pattern of the C-SELM area changes due to urban and suburban expansion, the total storage requirements also change.

Storage Types

Storage provided also reflects the different land use categories, and is divided in the following manner. The City of Chicago and several adjacent suburbs are presently served by combined sewer systems. These systems are served by the Chicago Underflow Plan. Other C-SELM suburbs and towns, including these which also have combined sewer systems, have storage provide by shallow pits or mined storage.

Areas which are in rural use now, but will be suburbanizing through the year 1990, will display two distinctly different types of storage. Both types will be discussed under the appropriate sub-section in this discussion. Rural areas possess an integrated storm-water management system, including storage, which is fully described in the subsection on rural stormwater management systems. Each of the different types of storage will now be described more fully.

Urban storage. The City of Chicago has long recognized the problem of sewer-overload during heavy, or prolonged, rainfalls and has helped to prepare a plan to solve the problem. The recently approved Chicago Underflow Plan[1] is the embodiment of a series of former studies of the Chicago combined sewer overloads. Harza Engineering Company and Bauer Engineering, Inc., both of Chicago, advanced the original plan for the use of deep tunnels for the convey-ance and storage of stormwater runoff. The C-SELM study has adopted the Chicago Underflow Plan for the C-SELM urban area. This ambiti-ous plan has a capacity to capture the runoff from all storms of re-cord except for three or four of the most severe ones. The overflow which is captured is stored temporarily, then pumped for treatment and eventual release to the waterway system. The system consists of

the following elements: conveyance tunnels, storage reservoirs and aeration systems, dewatering facilities, and solids management systems.

The conveyance tunnels are described in Appendix B, Section IV-E. The main storage site for the underflow plan is to be located in the McCook-Summit area on land now occupied by sludge storage lagoons belonging to the Metropolitan Sanitary District of Greater Chicago. After removal of the existing sludge, the site will be enlarged by excavation of the underlying rock by quarry methods. The resulting reservoir is to be 330 feet deep, 500 to 1,200 feet wide and almost 2.5 miles long. The total storage obtained in the reservoir is 57,000 acre-feet below Elevation, -100 (Chicago City Datum). There are two other reservoirs in the system. One is located in the vicinity of the proposed MSD O'Hare Treatment Plant and has a storage capacity of 1,800 acre-feet. The other is the existing Stearns Quarry, in the vicinity of 28th and Halsted Streets, and has a capacity of approximately 4,000 acre-feet. This storage will be used only during the most severe storms to flatten out the peak discharge period.

Aeration is provided at the Main Reservoir in the McCook-Summit area and at the O'Hare site. The Stearns Quarry reservoir is used only during large storms, and stores only highly diluted influent. No aeration is provided.

Aerators provided in the aeration system are the floating type. They are mounted on sliding carriage units, mounted in guide channels and supported from two walls of the reservoir. The carriage units and guide channels allow a vertical differential movement of over 200 feet. They are also provided with screens to prevent ice from entering and damaging the blades. The aeration provided is considered adequate to keep the wastewater in an odorless, aerobic condition.

In order to treat the captured combined sewer overflows in the nearby treatment plants, the reservoirs will be dewatered by underground pumping stations. The main reservoir will have such a station at its northeastern end, consisting of an intake conduit, a flap gate control chamber and bar screens and eight pumps, each capable of pumping 300 cfs. Discharge from the pumps will enter the manifold conduit with a tailer gate, and eventually a pressurized tunnel which will lead to the West-Southwest Sewage Treatment Plant in the treatment plant alternative or the respective access point in the land

treatment alternatives. The O'Hare reservoir is dewatered in a similar fashion and the effluent delivered for treatment to the O'Hare plant in the treatment plant alternative or the respective access point in the land treatment alternatives. Stearns Quarry is dewatered by gravity into the conveyance tunnel and does not have pumping facilities.

The sludge generated by settling in the storage reservoirs is estimated to reach a value of 670,000 cubic yards per year. Hydraulic dredge equipment is employed to clean the basins. The sludge is pumped through flexible floating pipelines connected to fixed pipe systems. The fixed header feeds a sludge pumping facility in the main pumping station. The sludge is conveyed to the treatment plant, or access point. Debris, which cannot be handled by the hydraulic dredge, will be periodically removed by other means when the facility is completely dewatered.

The general configuration of the Chicago Underflow Plan is shown on Figure B-IV-D-11.

Suburban storage. Suburban storage consists of mined and shallow pit components.

Mined storage

Urban or suburban areas other than the City of Chicago, such as Gary, Downers Grove or Wheaton, with combined sewer systems, are provided with suitable storage space. Consideration for such factors as high population density, septicity of effluent, aesthetics, and lack of suitable open space, is instrumental in the decision to construct these storage sites as mined storage. There are a total of 18 such sites scattered throughout the C-SELM area. Mining for such storages is conducted in either the Galena or Niagaran formations by the room and pillar method. The rooms so constructed would have a width of about 40 feet and a height extending to 50 feet or more. Unexcavated pillars, left in place between the rooms, in the Galena formation may be 120 feet wide, whereas those left in the Niagaran will be 250 feet on each side of the room. Aeration facilities are provided to prevent the stored

Figure B-IV-D-11
CHICAGO UNDERFLOW PLAN

O STORAGE
RESERVOIRS

□ TREATMENT
WORKS
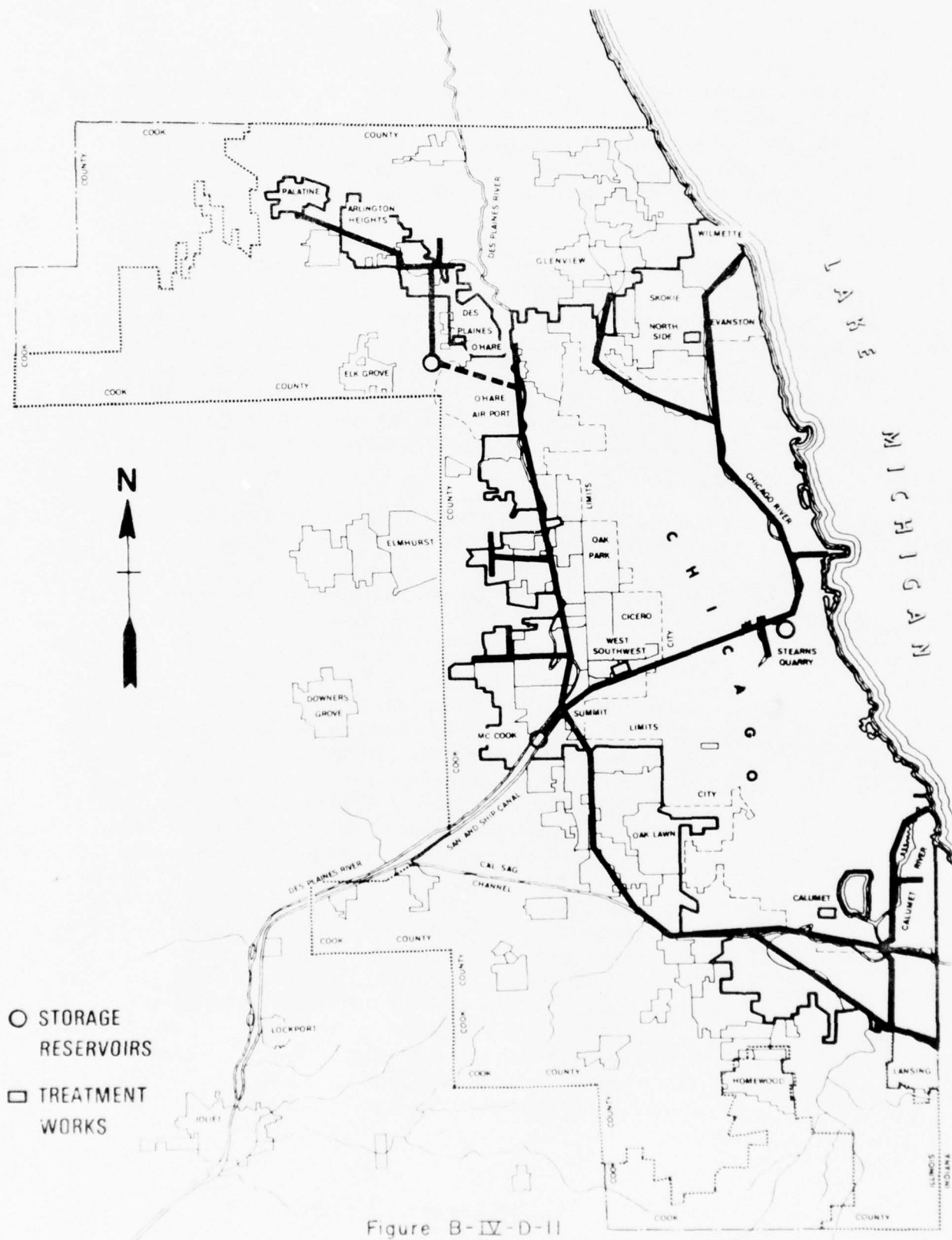
water from becoming anaerobic. The Niagaran formation
is one member of the shallow aquifer of the region.
The Galena-Plattville dolomite is a member of the deeper
aquifer system, but is relatively impermeable. To pre-
serve the quantity and quality of the water in these
aquifers and to protect it from possible contamination
by wastewaters stored in mined storages, aquifer pro-
tection systems must be provided. The detailed de-
scription of these systems is beyond the scope of this
report, but selection of regions of low permeability and
the use of artificial recharge techniques would be the
principal elements of any such system. A pumping
station associated with the mined storage, evacuates
the wastewater and moves it to nearby treatment facil-
ity or conveyance system.

Shallow pit storage

Other existing suburban areas, considered to be on
separate sewer systems, are served by shallow pit
storage. An extensive search for suitable sites has
been conducted through the available topographic maps,
and 77 possible locations have been selected. These
locations are tentative and subject to change should
circumstances warrant at the time of a more detailed
consideration. The sites were selected in naturally
low areas or existing quarries. The locations of mined
storage and shallow pit storage sites are presented on
Figure B-IV-D-12.

The shallow pits vary in surface area from 20 to 100
acres, depending on the contributory stormwater service
area. The design depth of storage is 15 to 20 feet
with 5 feet of freeboard provided. The side slopes of
the pit are maintained at a 4:1 ratio so that there are
no abrupt changes of elevation. Typical cross sections
of shallow pit storage with and without the permanent
pool are shown in Figure B-IV-D-13.

Some pits can be drained totally so that the open space
provided can be utilized for play fields and related activ-
ities. Those pits are equipped with an underdrainage
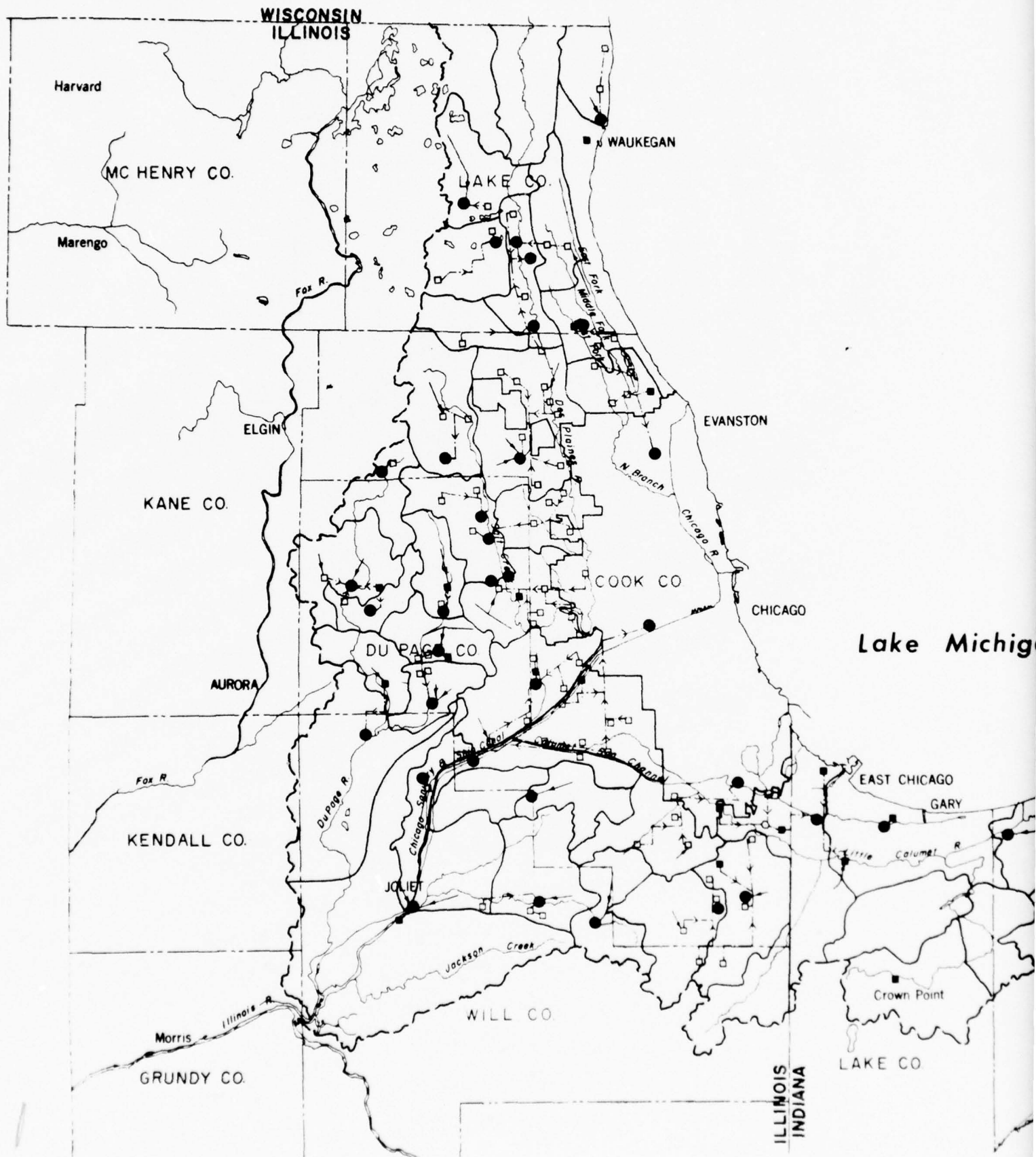
LEGEND

⌒⌒ STORM WATER SERVICE AREA BOUNDARY

–·–▸–·– EXISTING REGULATED SUBURBAN
STORMWATER CONVEYANCE SYSTEM

——▸ FUTURE (1970-1990) REGULATED SUBURBAN
STORMWATER CONVEYANCE SYSTEM

● TREATMENT FACILITY OR ACCESS POINT

□ SURFACE STORMWATER STORAGE

■ DEEP PIT STORMWATER STORAGE

○ STORMWATER BOOSTER PUMPING STATION

Figure B-Ⅳ-D-12

LOCATIONS OF MINED STORAGE
AND SHALLOW PITS

B-Ⅳ-D-31

PUMP STATION

TO TREATMENT

TREE AND SHRUB BUFFER

FENCE

20'

15' STORMWATER STORAGE

1
4

1
4
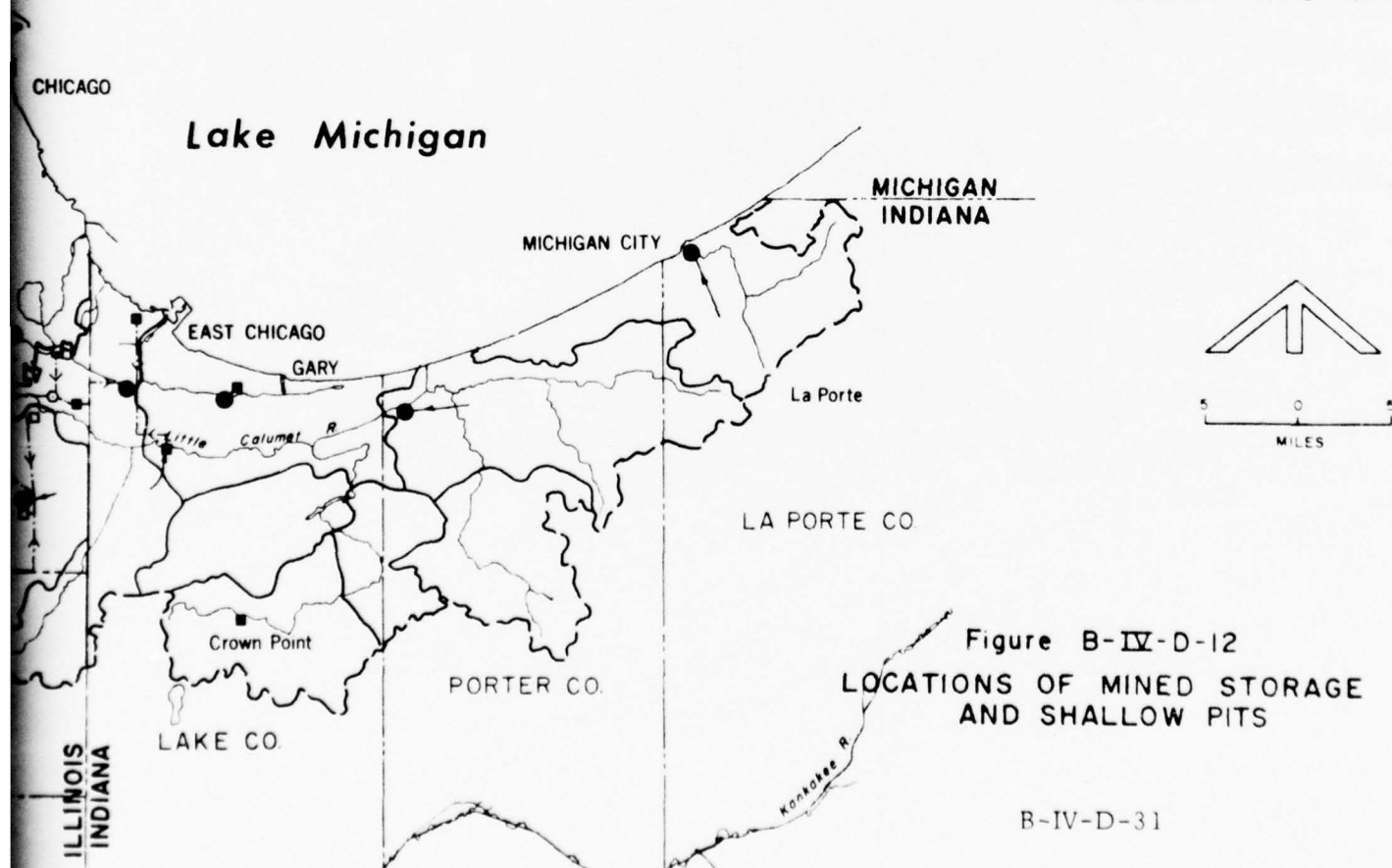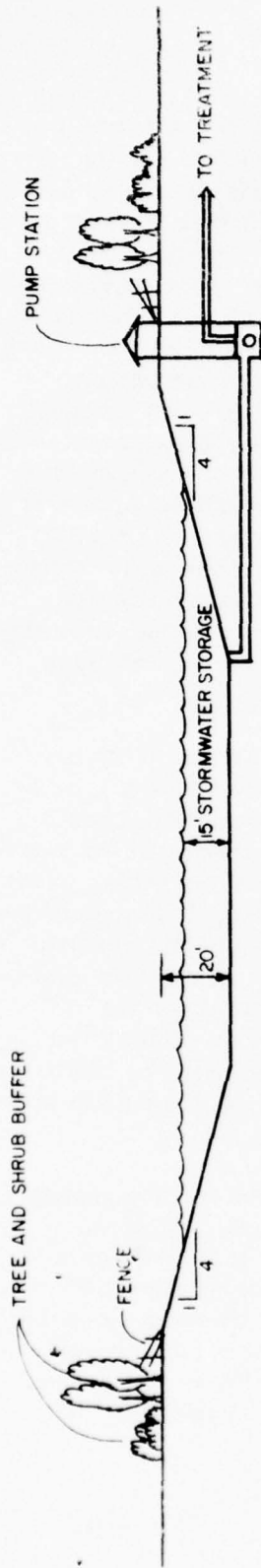
SHALLOW PIT - NO PERMANENT POOL

TO TREATMENT

5'

15' STORMWATER STORAGE

5'-10'

PERMANENT POOL

1
4

1
4
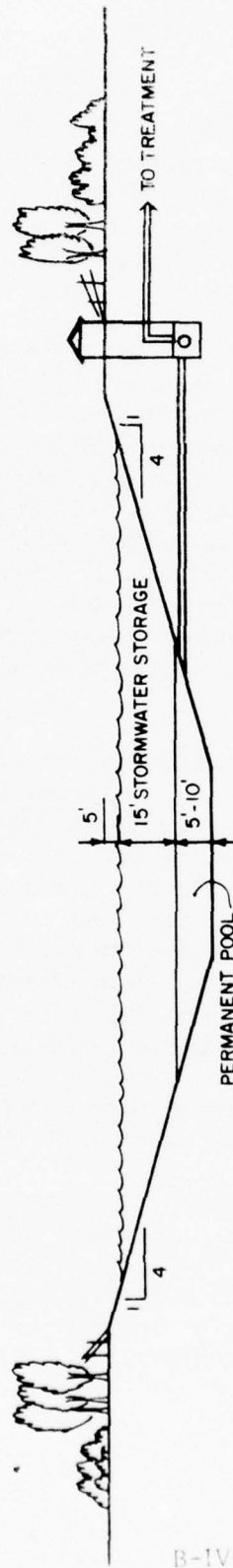
SHALLOW PIT WITH PERMANENT POOL

Figure B-IV-D-13
SHALLOW STORAGE TYPES

system, so that drying of the bottom of the pit may proceed rapidly, and permit early use of the pit for activities described previously. The pits may also be provided with air-bubbler aeration systems to permit an odorless, aerobic condition to exist in the pit during the presence of stormwater. The system thus envisioned consists of a system of buried lines with air nozzles protruding just above the surface of the pit bottom. An air compressor is turned on when the stormwater begins to arrive, and continues to operate for as long as the need for aeration exists. Some pits may be excavated deeper to provide permanent pools of water with a minimum depth of 5 to 10 feet. These pits would serve as water recreation areas, even though swimming or water skiing would not be recommended. Although only stormwater enters the shallow pits, it is nevertheless contaminated. Aeration facilities, similar to those described with drainable pits, are provided with permanent pool pits.

Stormwater from the contributory drainage basin is collected and delivered by gravity to the shallow pit storage. Prior to entering the storage, flows will pass through the grit removal chambers to screen out as much settleable matter and large floatables as possible. Grit removal chambers will have front-cleaning heavy-duty bar screens with transfer conveyors for continuous removal of debris. The housing for bar screen and debris disposal area will be separated from the shallow pit proper by well-planned natural barriers to protect the aesthetics of the site. Grit and debris will be deposited into dump truck containers for removal to a landfill site or solid waste management site for ground cover.

Evacuation from storage is accomplished by a pumping station delivering the effluent at a design pump-out rate, to the nearest treatment facility or conveyance system access point. Each shallow pit storage is equipped with an emergency overflow structure, capable of discharging flows in excess of design capacity of the storage. These infrequent excess flow spills are directed to nearby waterways. Excess flows are an

integral part of the overall design parameter consideration of the management system and are expected to occur between 1 to 5 times in a projected period of 21 years.

Buffer zones. Provision is made for a buffer zone around each pit so that proper landscaping may blend the pit into the surrounding suburban environment. It is visualized that the periphery of the storage may be planted with bushes and trees to screen it from the surrounding residential areas.

Time available for recreation. The availability of space in the pits for purposes other than storage of stormwater was studied. The study encompasses the stream basins of the Des Plaines River, Salt Creek, North Branch of the Chicago River, and Thorn Creek. Total available dry time in the 19-21 years of analyzed record, for months of maximum outdoor activity is as follows: June = 38%, July = 53%, August = 78% and September = 89%. No allowance is made here for drying time required to permit the soil to bear traffic. Figure B-IV-D-14 shows per cent of dry time on a monthly basis for the entire year for the four streams investigated as well as for an average basin curve. The study reveals that although dry time is available throughout the entire year, most of it occurs during the fall and winter. Football, ice skating, and other winter sports would be well accommodated.

Storage in suburbanizing areas. Areas which are at present in rural land use, and which are projected to be in suburban use by 1990, are provided with storage in two distinctly different ways. These are: conversion of rural stormwater retention basins to suburban shallow pit storage, and development of surface ponds by developers to satisfy the new ordinance passed by the Metropolitan Sanitary District of Greater Chicago.
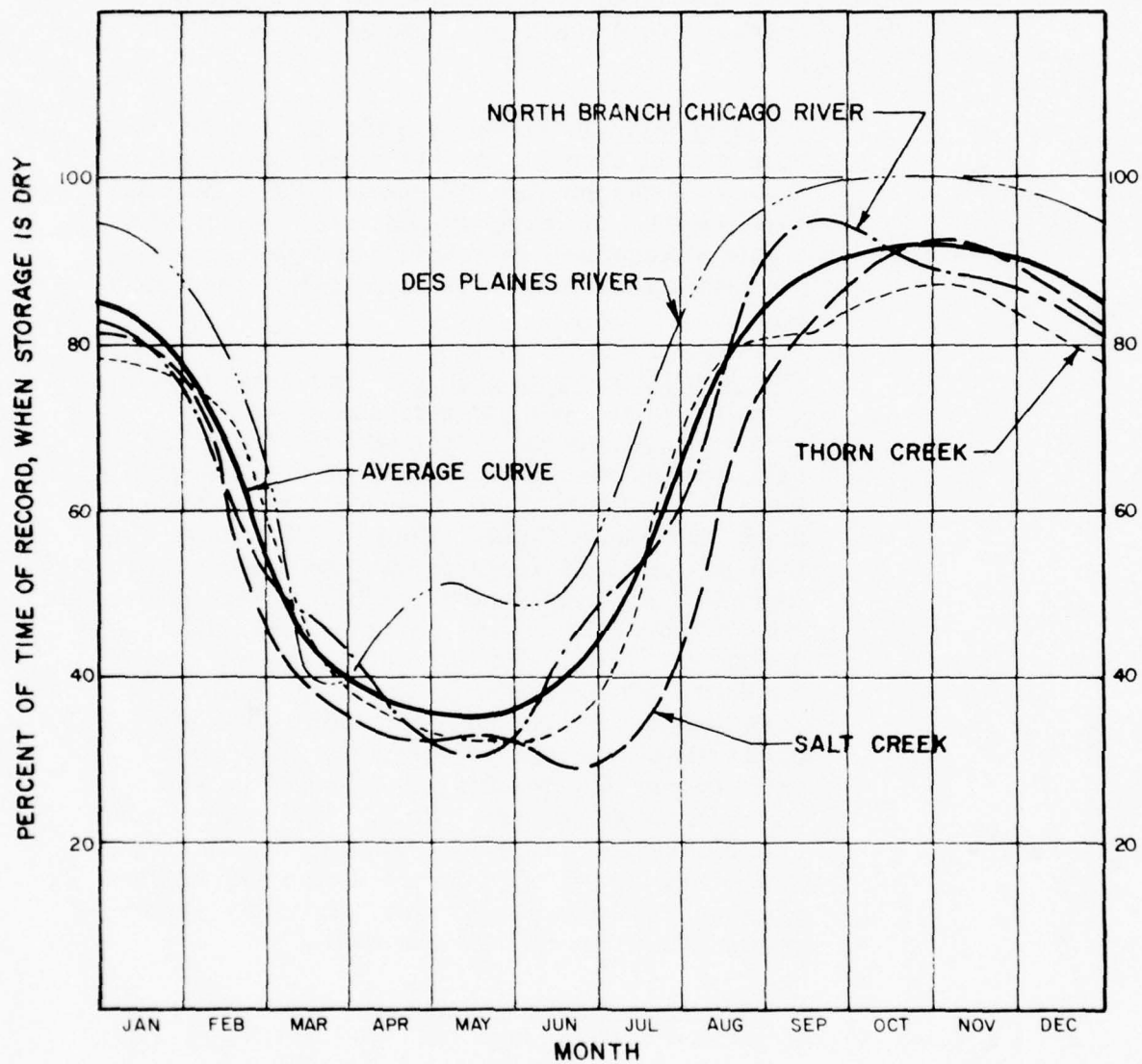
Figure B-Ⅳ-D-14

PERCENT OF STORAGE DRY TIME FOR
VARIOUS DRAINAGE BASINS IN C-SELM AREA

B-IV-D-5

This two-pronged approach is prompted by the fact that some areas will be developing in the immediate future and, consequently, will not have the benefit of previously-developed rural stormwater management sites to build upon. Private storage facilities are already being used in recent land developments. For example, one of the developments in the Arlington Heights area has a pond of approximately three acres in surface area and ten feet in depth. The pond is connected to the stormwater collection system and drains by pumpage to Salt Creek. When the regional wastewater management system becomes a reality, and is fully implemented, all of these ponds will be connected to the nearest treatment plant or conveyance system.

The conversion of rural stormwater retention basins to suburban shallow pits is proposed when a particular rural drainage basin becomes suburbanized. In this case, the land treatment system which is an inherent part of the site will be abandoned. It would be the developer's responsibility to blend his plans into the existing overland drainage system of grassed waterways. In so doing, he will avoid extensive and expensive storm sewer development and will preserve the natural beauty of open space provided in the grassed waterways. The stormwater retention basin will be connected by a pumping station and pressure line to the nearest treatment plant site or conveyance system. Underdrainage, aeration, and grit removal systems may be installed. Description of the suburban shallow pit storage discusses these installation in more detail. Figure B-IV-D-15 shows the different storage systems depending on land use.

The means of transportation of stormwater to the storage system is discussed in the following section.
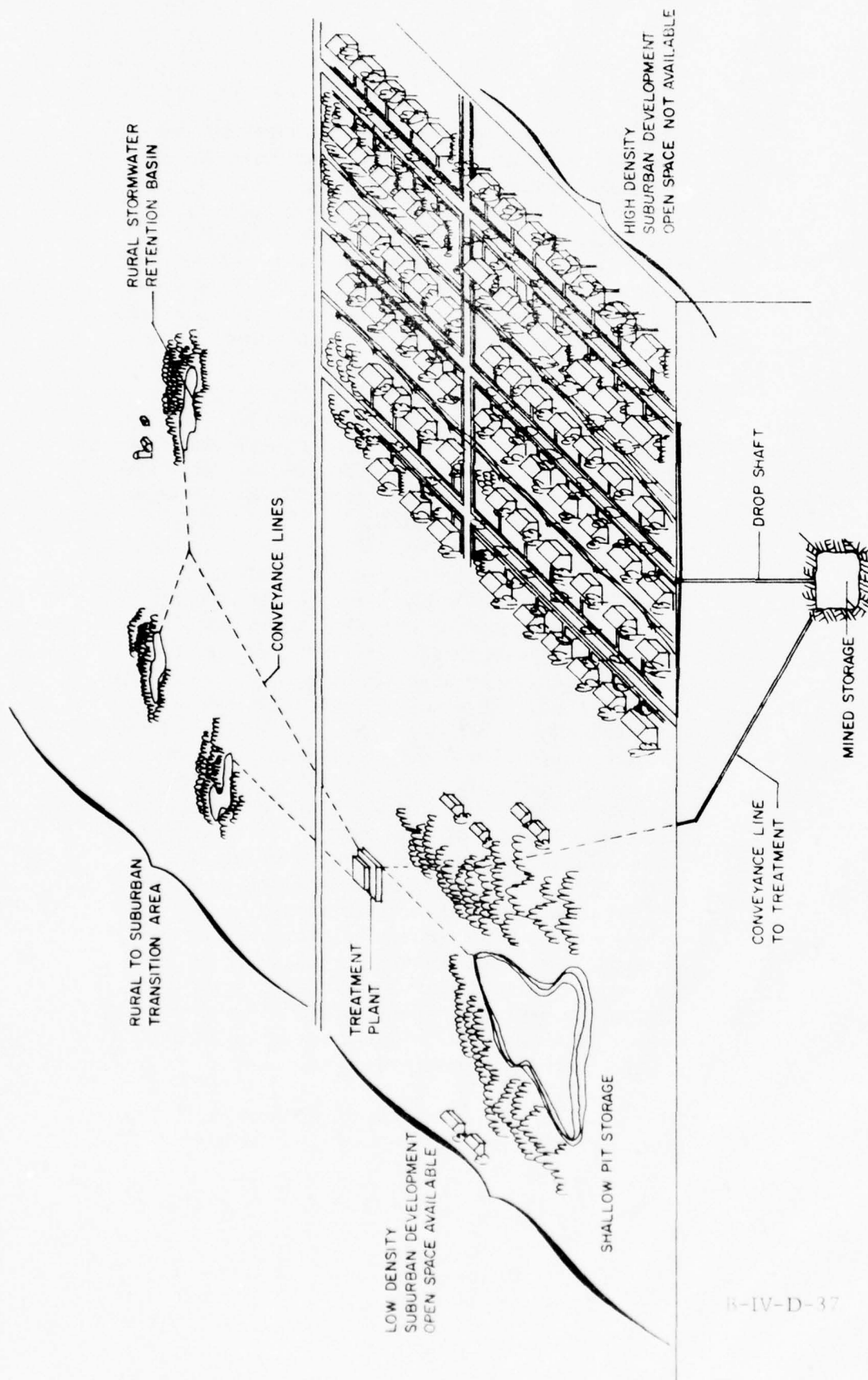
Figure B-IV-D-15

DIVERSE STORAGE SYSTEMS DEPENDENT ON LAND USE

COLLECTION SYSTEMS

Design Philosophy

Collection systems, as defined for the purposes of the C-SELM wastewater management system study, collect unregulated stormwater, or dry-weather municipal flows, and transport these flows to storage, or access points for treatment facilities respectively. Collection systems are divided into three categories depending on land use classification, i.e., urban, suburban and rural. Each of these categories is briefly discussed.

Urban Collection

This collection system is basically the combined sewer system of the City of Chicago and related suburbs with similar combined sewer systems. For the most part, this system is in existence, and collects municipal and industrial flows on a dry-weather basis, with added flows of stormwater on an intermittent or storm-related basis. The interconnection between the urban collection system and the proposed Chicago Underflow Plan storage system is accomplished via the collection sewers, which intercept some 640 outfall points and transport the flows to 341 drop shafts leading to a tunnel system and, eventually, to storage. Tunnels are constructed in the Silurian Dolomite rock formations 150 to 200 feet below the surface of the waterways. Total length of the tunnel system will be some 120 miles, and the diameter of the tunnels will vary from 10 to 42 feet. Tunnels up to 35-feet in diameter are drilled by mole mining machines and those of larger diameter by drill and blast method. The total volume provided by the tunnels is 9,100 acre-feet. Physically, the tunnel system emanates from the main storage site at the McCook-Summit area and will split into three main branches. The Des Plaines branch extends northward along the Des Plaines River to the Village of Des Plaines and thence northwest as far as the Village of Palatine. The Mainstream Tunnel extends along the Sanitary and Ship Canal, the north and south Branches of the Chicago River and the North Shore Channel to the Wilmette control works. Finally, the Calumet Tunnel System follows the public right-of-way in the southeasterly direction to the Sag Channel, then eastward under the Little Calumet, Grand Calumet and Calumet River. The cost of the interceptor sewers and the drop shafts is included in the total cost of the Chicago Underflow Plan.

Suburban Collection

The collection systems of municipal and industrial flows, presently in existence, terminate at treatment plants. Upon regionalization of the treatment plant systems, abandoned treatment plants will become access points for the existing collection systems. The flows from the abandoned plants will be conveyed to regionalized plants via a conveyance system. This system is discussed in Appendix B, Section IV-E. Generally, all the existing suburban stormwater collection systems presently terminate at the streambeds. With the installation of shallow pit storages throughout the C-SELM area it is necessary to intercept all the stormwater collection systems and direct the flows to the respective storage pits. It is proposed that local authorities have the responsibility to extend the transport of all existing stormwater flows within their jurisdiction, to the assigned shallow pit, or mined storage units.

Rural Collection

Rural stormwater runoff is collected via overland flows to grassed waterways. The system of grassed waterways terminates in the interceptor waterways which generally parallel the local streams, and which empty their flows into the retention basins. The collection of rural stormwater is thus accomplished through this natural means rather than through a system of man-made sewer pipes. A very detailed description of the rural stormwater collection system is contained in this Appendix in the section on 'Rural Stormwater Management System'.

RURAL STORMWATER MANAGEMENT SYSTEM

Introduction

As was pointed out in Appendix B, Section II-D, the overall concept of rural stormwater runoff management includes quality and quantity management of all runoff as well as management of the land resource. With this in mind, the management system starts with a comprehensive land management and soil conservation system with the purpose of reducing flow quantities and sediment loadings. Water which does run off is channeled as overland flow to retention basins. From the retention basins, the flows are conveyed to spray-irrigation

sites which employ center-pivot spray irrigation machines for application. Renovated flows are collected by a plastic pipe drainage system. This drainage system functions by gravity and releases flows to the nearest natural water course.

One of the aims of the analysis presented in the following sections is the determination of what might be called a standard design module. Towards this end, three watershed areas within the designated rural area were selected and studied following the philosophy of design presented in Appendix B, Section II-D, using subwatersheds delineated within the framework of each larger watershed. The application of the design philosophy to these subwatersheds is covered in the following sections. The land-management system is discussed first since it has universal application to all watershed areas. This is followed by a discussion of key watershed characteristics and how they impact the overall system design. Integrated into this discussion on watershed characteristics is a discussion of retention basin configuration. Following this discussion is a point-by-point basis of design discussion for each main system component.

Land Management Considerations

Current practices in land management. Soil erosion and sediment control are some of the main concerns of the Soil Conservation Service (SCS). Practices which prevent erosion are beneficial not only to farmers but also to those who live in the watershed area downstream. Elimination of tons of sediment carried away by overland runoff not only improves the quality of the receiving stream, but also preserves one of our most valuable natural resources, agricultural topsoil. It is important, therefore, to recognize the practices recommended by the SCS and to implement them where possible in the course of the rural stormwater management study.

SCS recommendations concern tillage practices, crop rotation programs, ground cover considerations and drainage control. All practices will not be detailed here, but those specific ones used in the land management are discussed below.

Recommended land management package. Discussion in Section II presented the philosophy behind the need for good land management practices. Of primary concern is the infiltration of larger quantities of rainfall into the soil. Towards this end, two

techniques will be addressed: (1) tillage operations and (2) crop residue management.

Tillage operations for seedbed preparation should be kept at a minimum, thus enhancing the roughness of the soil surface and increasing the available storage. The more a seedbed is worked and the more cultivation takes place, the more the soil structure is broken down. This tends to destroy the natural aggregation of the various soil particles and subsequently decreases the intake rate of the soil, the unsaturated rate of percolation, and the saturated permeability. (See Annex on Soil Information, Soil Characteristics, and Site Selection for additional information on the importance of these criteria.) In addition, less tillage reduces the susceptibility of soils to erosional factors from not only water, but also wind.

Seven major operations are currently advanced as practical, economical methods of providing less tillage. These are covered in detail in the Annex section on Land as a Method of Treating Wastewater. They are: (1) no-tillage, (2) strip-till planting, (3) combined tillage, (4) chisel plow, (5) heavy-duty disc, (6) field cultivator, and (7) fluted coulter. The no-tillage method is recommended for application here.

In this method, a planter is used with a rolling coulter, which cuts a slice in the soil for a narrow planter-runner. A weighted wheel runs behind this runner and pushes the soil back into contact with the seed.

In addition to this no-tillage operation, crop residues from the harvesting operations are to be left where they fall. This action also increases the soil's water-holding capacity, and is particularly important during the nongrowing period, when the otherwise bare soil is susceptible to rainfall impact packing, and freeze-thaw conditions which enhance erosional possibilities. The large void, or pore, spaces within this surface mulch provides instantaneous surface storage and allow movement directly into the soil medium before the water has a chance to run off.

Any runoff which is produced on properly managed areas will have a much lighter load of sediment. It has been projected that proper management techniques, such as advanced above, will reduce sediment loads from around 12 to 15 tons per acre per year to as low as 0.75 to 1.0 tons per acre per year.

Appendix B, Section II-D also presented the concept of a collection system incorporated within the framework of natural drainage conditions. This collection system would transport flows overland to a retention basin for temporary storage. The natural drainage system is a very important link in the management of the watershed.

Existing, natural drainage ways are sculptured by following the natural pattern of flow. Watershed areas and gullies are filled in. A waterway is shaped in a shallow V with gentle side slopes (4:1) to carry a large flow of water and at the same time to allow easy access to farming implements. A plastic pipe drainage line is run parallel to the notch of the waterway V, and from four to five feet below the surface to provide removal of any water that may infiltrate the grass cover. The grass cover is established through the planting of a healthy cover crop such as bermudagrass or buffalograss.

Slope considerations along the length of the waterway are important. Permissible slope varies with the type of grass cover and, of course, the naturally occurring slope of land in question. For the two types of grass mentioned above, permissible velocity have been established based upon the prevailing slopes. Representative velocities are shown in the following Table B-IV-D-2.[5] In order to move maximum quantities of water with least possible cross-sectional areas, highest permissible velocities are desired. To obtain these velocities, with no danger of erosion, slopes are selected which limit the highest design velocity to permissible values. For example, for bermudagrass, with a slope range between zero and five percent, the maximum permissible velocity is six feet per second.

To accomplish uniform slope over the length of any grassed waterway, and therefore a maximum permissible velocity and optimum capacity, control structures are needed. A design slope of two percent was established and control structures were placed at needed locations along the run of the grassed waterway to maintain this slope. Exact placement of structures is dependent upon the configuration of the particular area under consideration.

Figure B-IV-D-16 presents a conceptual overlay of the recommended land management package adapted to an actual reach of waterway within the C-SELM study area. The recommended land management

Table B-IV-D-2

PERMISSIBLE VELOCITIES FOR
CHANNELS LINED WITH VEGETATION[a,b]

| Cover | Slope Range[c] | Permissible Velocity | |
		Erosion re-sistant soils	Easily Eroded Soils
	Percent	Ft. Per Sec.	Ft. Per Sec.
	0-5	8	6
Bermudagrass	5-10	7	5
	over 10	6	4
Buffalograss	0-5	7	5
Kentucky bluegrass	5-10	6	4
Smooth brome	over 10	5	3
Blue grama			
	0-5[c]	5	4
Grass mixture	5-10	4	3
Lespedeza sericea			
Weeping lovegrass			
Yellow bluestem			
Kudzu	0-5[d]	3.5	2.5
Alfalfa			
Crabgrass			
Common lespedeza[e]	0-5[f]	3.5	2.5
Sundangrass[e]			

[a]Source: U.S. Soil Conservation Service

[b]Use velocities exceeding 5 feet per second only where good covers and proper maintenance can be obtained.

[c]Do not use on slopes steeper than 10 percent except for side slopes in a combination channel.

[d]Do not use on slopes steeper than 5 percent except for side slopes in a combination channel.

[e]Annuals--used on mild slopes or as temporary protection until permanent covers are established.

[f]Use on slopes steeper than 5 percent is not recommended.

LEGEND

〰〰〰 PERENNIAL DRAINAGE

〰·〰·〰 INTERMITTENT DRAINAGE

⟨⟩ GRASSED WATERWAY

▢ DROP STRUCTURE

○──▶── DRAINAGE TILE

SCALE OF FEET

Figure B-IV-D-16

RECOMMENDED MANAGEMENT PACKAGE

system described above is applicable to all area designated as rural within the study area. The management techniques are of such a general nature that this universal application is feasible. Areas not in agricultural land use, such as pasture or forested areas are managed by the placement of grassed waterways and other erosion-resistant maintenance techniques.

Management System Layout Considerations

General. As mentioned above, three representative watersheds were selected for detailed analysis. Before information is presented on component design, a brief discussion of general management system layout and its impact on the subsequent overall system design is presented.

Watershed topographic characteristics. One factor which plays a key part in the management of stormwater runoff is the topographic nature of drainage area being considered. The topography effects not only the collection and conveyance of stormwater flows, but also the configuration of the retention basins. In addition, it effects the location and size of any spray irrigation areas.

In areas of little topographic relief, grassed waterways for collection and conveyance are easily constructed and maintained. Retention reservoirs would commonly be either excavated or diked construction. See Figure B-IV-D-17. This type of structure is usually more expensive than the conventional valley-type structure. The location of center-pivot irrigation rigs would be simplified, with the possibility of greater rig length and subsequent savings.

On terrain with more topographic relief, valley-type retention basins are applicable and usually more desirable because of construction cost savings. Grassed waterway collection and conveyance units can be constructed on steeper grades, requiring high control structure costs to hold flow velocities within permissible values. Irrigation site areas would be smaller and more difficult to locate, and more numerous, with resulting higher cost for the treatment phase of the rural system.

Selected watersheds. The United States Geological Survey (USGS) topographical sheets, USGS Flood Hazard Maps, available aerial photos, and information from the Soil Conservation Service (SCS)

FLOODWATER STORAGE ZONE
(FILLED BY GRAVITY, EMPTIED BY GRAVITY)

EARTH DAM

PIPE OR SPILLWAY

RECREATION POOL

VALLEY STORAGE *

FLOODWATER STORAGE ZONE
(FILLED BY PUMPS; EMPTIED BY GRAVITY)

EARTH DIKE

RECREATION POOL

PUMP STORAGE–UPGROUND RESERVOIR *

ORIGINAL STREAM

FLOODWATER STORAGE ZONE
(FILLED BY GRAVITY, EMPTIED BY PUMP)

RECREATION POOL

PUMP STORAGE–EXCAVATED RESERVOIR *
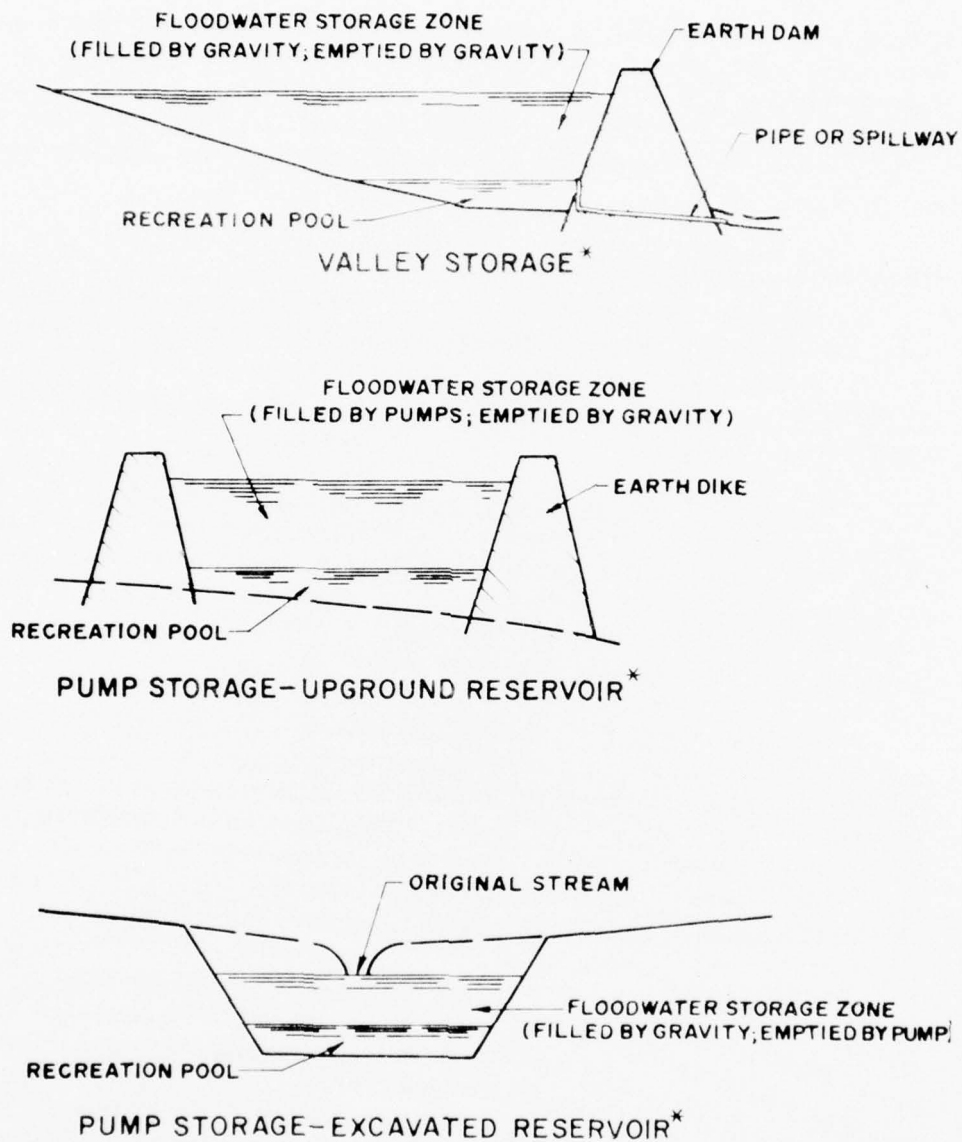
* NOT TO SCALE, EXAGGERATED VERTICAL SCALE

Figure B-IV-D-17

TYPES OF RURAL STORMWATER RETENTION AREAS

county soil surveys were employed to choose three representative watershed areas. The following watershed or partial watershed basins were selected: Mill Creek, Lake County, Illinois; Jackson Creek, Will County Illinois; and Salt Creek, Porter County, Indiana. The general locations of these basins are shown in Figure B-IV-D-18. Each watershed area is divided into subwatershed areas, based upon naturally ocurring drainage patterns.

Watershed drainage patterns and the siting of retention basins. Three distinctive drainage patterns were observed in the subwatersheds. The first might be identified as a branch drainage pattern, where overland flow reaches small intermittent drainage channels, from which it is conveyed to a single main channel. For the watersheds observed, this type of drainage pattern is usually associated with headwater areas of the smaller tributary streams. Management of this type of drainage pattern is accomplished by placing the retention basin across the main channel and capturing runoff at the outflow of this smaller subwatershed area. The retention basin is located just upstream from the point where perennial flow is first observed. Runoff can easily be channeled to the low point by the grassed waterway system described in the land management discussion presented above.

A second subwatershed type is also commonly observed downstream of the branch type pattern discussed above. This drainage pattern usually developed as a series of parallel channels entering a main perennial stream. In most cases, each of these small intermittent channels serves a drainage area that is much too small to be managed on an individual basis, but, nevertheless, could not be ignored. Therefore, they are integrated into larger subwatershed areas by placing grassed, interceptor waterways parallel to the main stream to intercept the outflow from them, and to convey it downstream to the retention basin which is located off-stream. This allows the main channel of the perennial stream to remain free flowing and unpolluted by stormwater flows.

One variation of this management system deals with a parallel series of small drainage channels on both sides of a main perennial stream. In this situation, stormwater flow is conveyed to the opposite side of the stream to the off-stream retention basin. This is accomplished either by pumping or by siphon.
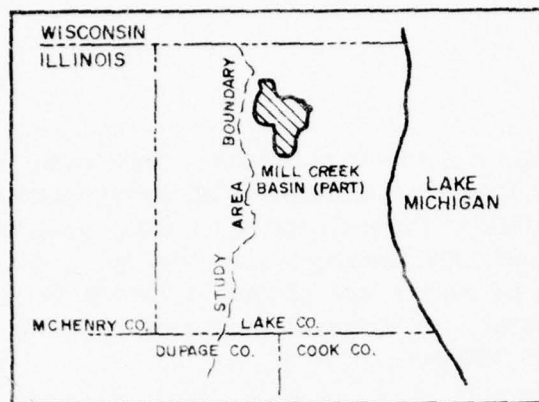
A) MILL CREEK, LAKE COUNTY, ILLINOIS
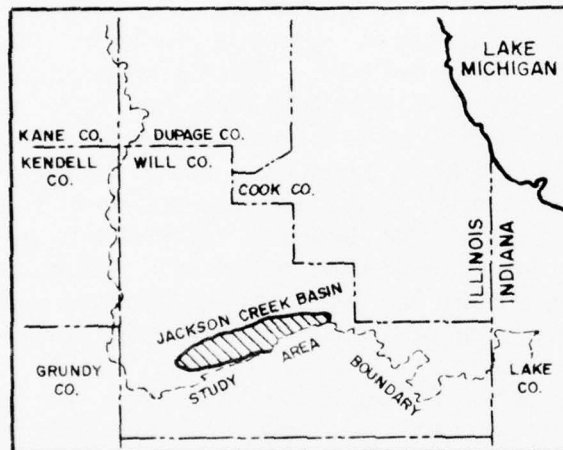
B) JACKSON CREEK, WILL COUNTY, ILLINOIS
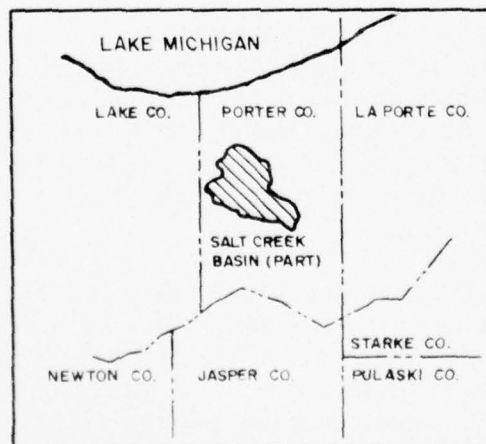
C) SALT CREEK, PORTER COUNTY, INDIANA

Figure B-IV-D-18
REPRESENTATIVE RURAL WATERSHEDS

The third subwatershed type observed contains depressional areas associated with undeveloped topography. This depressional drainage pattern is not topographically connected with the surrounding drainage patterns. Flow to these depressional areas is managed in two ways. If the drainage area is small, stormwater flows are simply ponded in the low spot of the depressional area and allowed to infiltrate into the groundwater, possibly aided by a pipe drainage system where necessary. If the drainage area is large, stormwater flows are ponded, and then pumped across or drained by gravity to other drainage areas tributary to grassed waterways leading to storage. In some instances, such flows are routed directly through a pipe to the retention basin.

Care was taken to locate all retention basins where they do not cross major transportation corridors or cause relocation of any housing or commercial installations, and where they would fit tolerably into the natural topographic pattern.

Spray irrigation area site selection. Spray areas are located, when possible, on existing agricultural lands. In every case, land use for irrigation was either in an agricultural cropping or pasturing use. Every effort was made to keep the spray irrigation area within the confines of existing road and transportation patterns.

Soil considerations. The characteristics of the soil within the rural area were important for not only retention basin siting, but also for the irrigation area siting. The following discussion presents a general description of the soils found within the selected rural watersheds.

Lake County, Illinois: Mill Creek Basin.

Predominant soil associations within the Mill Creek representative rural management basin are; Morley-Markham-Houghton; Zurich-Gray-Wauconda; and Elliot-Markham.

Morley-Markham-Houghton are gently sloping to steep, well-drained to moderately-well-drained, deep soils that have moderately slow permeability (0.2 to 0.6 inches per hour); and level to depressional, very dark colored, very poorly drained organic soils. Morley

and Markham soils occupy the higher parts of the land-
scape. Both soils are well-drained. Markham soils
have a darker colored surface layer than Morley soils.
Both soils have a brown or dark yellowish-brown sub-
soil. Houghton soils are level or depressional organic
soils. Many of the closed depressions in which they
occur are not drained and are ponded year round.
These three soils are predominate in this rural manage-
ment area. They cover the entire western one-half of
the area, and most of the north one-half.

Zurich-Gray-Wauconda are nearly level to moderately
steep, well-drained to somewhat-poorly-drained, deep
soils that have moderate permeability (0.6 to 2.0 inches
per hour). Zurich and Gray soils are more sloping
than other soils in this association. Both are well
drained to moderately well drained. Gray soils have a
thicker surface layer than Zurich soils. Both have a
brown or dark yellowish-brown subsoil. Wauconda
soils are nearly level to gently sloping soils and are
somewhat poorly drained. They have a dark grayish-
brown to light olive brown, mottled subsoil. This soil
is principally located along both sides of the main
branch of Mill Creek.

Elliott-Markham are level to strongly sloping, well-
drained to somewhat-poorly-drained, deep soils that
have moderately slow permeability (0.2 to 0.6 inches
per hour). Elliott soils are gently sloping and some-
what poorly drained. They have a brown or dark grayish-
brown mottled subsoil. Markham soils are on the higher
parts of the landscape. They are well drained to mod-
erately well drained. They have a lighter colored sur-
face layer than the Elliott soils and have a brown or
dark yellowish-brown subsoil. These soils make up a
very small portion of the management area in the south-
eastern corner.

These soils do not appear in the Illinois drainage guide,
but by comparison with similar soils it can be assumed
that they are drainable through the use of drainage tiles.
Available information on the irrigation possibilities from

the Illinois Irrigation guide indicates that the soil can
be irrigated.

Will County, Illinois: Jackson Creek Basin

There is a single, predominant soil association which
covers the entire watershed area under consideration.
The soil association is Elliott-Ashkum. Elliott silt
loam is a dark, imperfect oxidized brunizem soil formed
in silty clay loam till with less than two feet of
medium-textured surficial drift (including loess). Water
holding capacity is high, but permeability in the lower
solum and underlying till is slow (.2 to .6 inches/
hour). Tiles function slowly but are generally effective.
Ashkum silty clay loam is a very dark, poorly-oxidized
soil formed in less than 42 inches of medium-to-
moderately fine textured drift on silty clay loam till.
Water capacity is high and permeability is moderate in
the solum (.6 to 2 inches per hour) but moderately slow
in the underlying till. Tile function slowly, but are
generally effective.

The Illinois drainage guide classes these two soils,
Elliott and Ashkum, as 3B, and 3A, respectively. These
soils are drainable, by tile, in these classifications.
The Illinois Irrigation guide also classifies both soils
as irrigable.

Porter County, Indiana: Salt Creek Basin

Major soil associations in this management area are the
Morley-Blount-Pewamo and the Riddles-Rawson-Morley.
There is a small area of the Rensselaer-Gilford associ-
ation.

Morley-Blount-Pewamo soils are nearly-level-to-steep,
poorly-to-well-drained upland soils on moderately fine
textured glacial till. Blount soils occur on slopes
ranging from 0 to 3 per cent. These soils are deep,
somewhat poorly drained, slowly permeable, and have
high available moisture capacities. Tiling is recom-
mended. Morley soils occupy slopes ranging from 2 to
25 per cent. These soils are deep, moderately well

drained and slowly permeable. Pewamo soils are deep, slowly permeable, very poorly drained and have high available moisture capacities. Tiling is recommended.

Riddle-Rawson-Morley are gently-sloping-to-moderately-steep, well-drained upland soils on medium-to-moderately fine textured glacial till.

These soils are not well suited to irrigation, but with well designed drainage systems they can be used for our purposes.

Summary

The general nature of the soils in the rural areas studied can be assumed to be much tighter than the soils under consideration for spray irrigation areas in the overall study. However, all indications show that the soils can be irrigated, and thus provide the planned treatment, if proper surface and subsurface drainage is provided, land-management of the spray areas is kept at a high level, and proper scheduling is maintained.

Conclusions. Figures B-IV-D-19, B-IV-D-20 and B-IV-D21 present the conceptual layouts of the three selected watersheds. The total number of subwatersheds delineated is 22. The location of all major system components are shown on these figures. Retention basin and spray irrigation sitings reflect the discussion presented above.

The discussion which follows presents general design information on management system components.

Component Design Considerations

Collection and conveyance. Collection and conveyance is accomplished through the use of natural drainage patterns. Flow is facilitated by the improvement of the channel as reflected in the discussion of the land management protocol presented in a preceding section. Interceptor grassed waterways are designed on the same basis as the overland grassed waterways. Subsurface drainage is provided at a uniform depth of five feet below the centerline of the grassed waterway. Four-inch plastic drainage pipe is used under
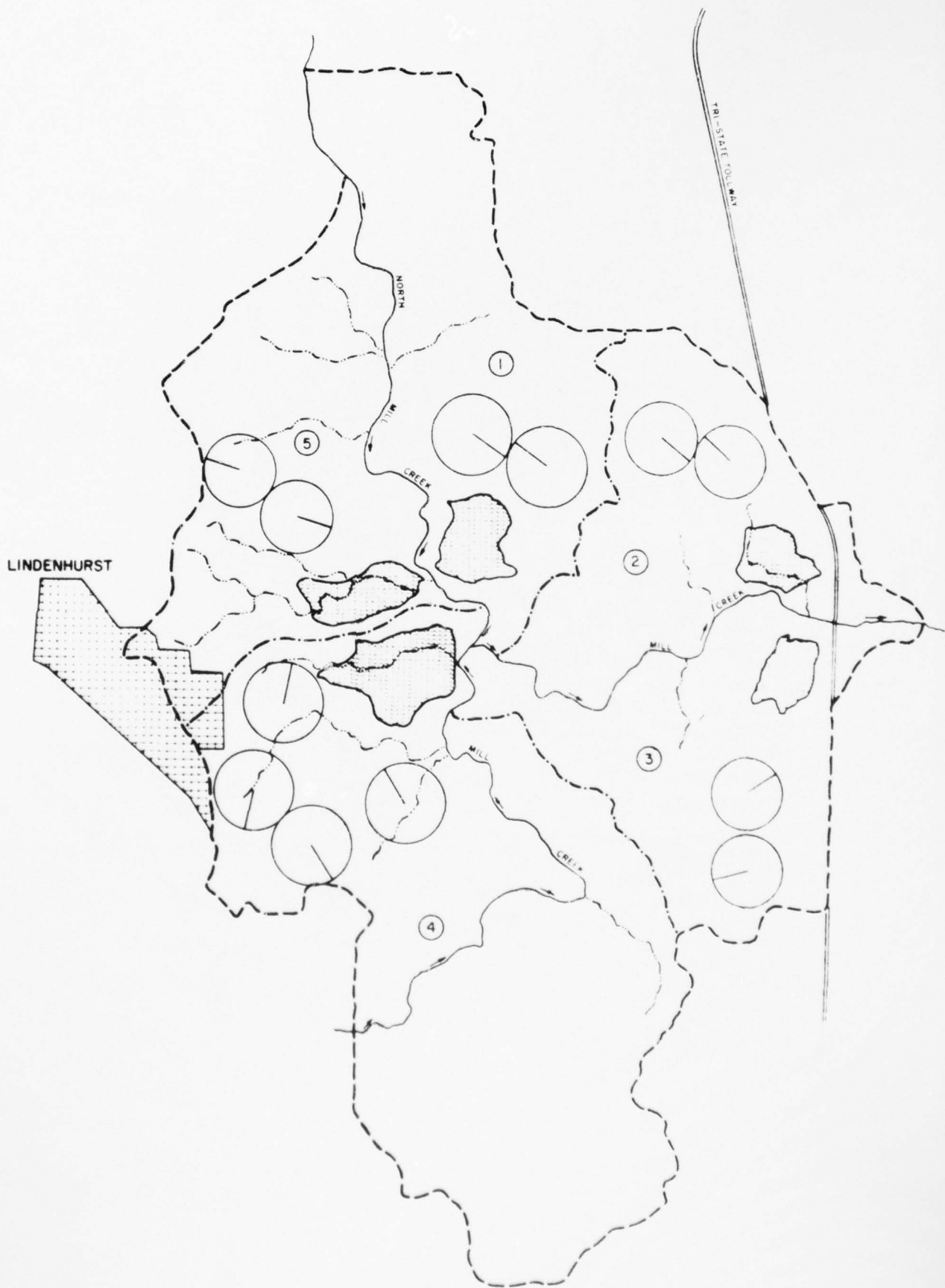
LINDENHURST

TRI-STATE TOLLWAY

CREEK

MILL

SCALE O

WISCONSIN
ILLINOIS

MILL CREEK
BASIN (PART)

MC HENRY CO   LAKE CO
DU PAGE CO COOK CO

STUDY AREA BOUNDARY

LOCATION PL

Figure

REPRESENTATIV
MILL CREEK, (PART) L

B-IV
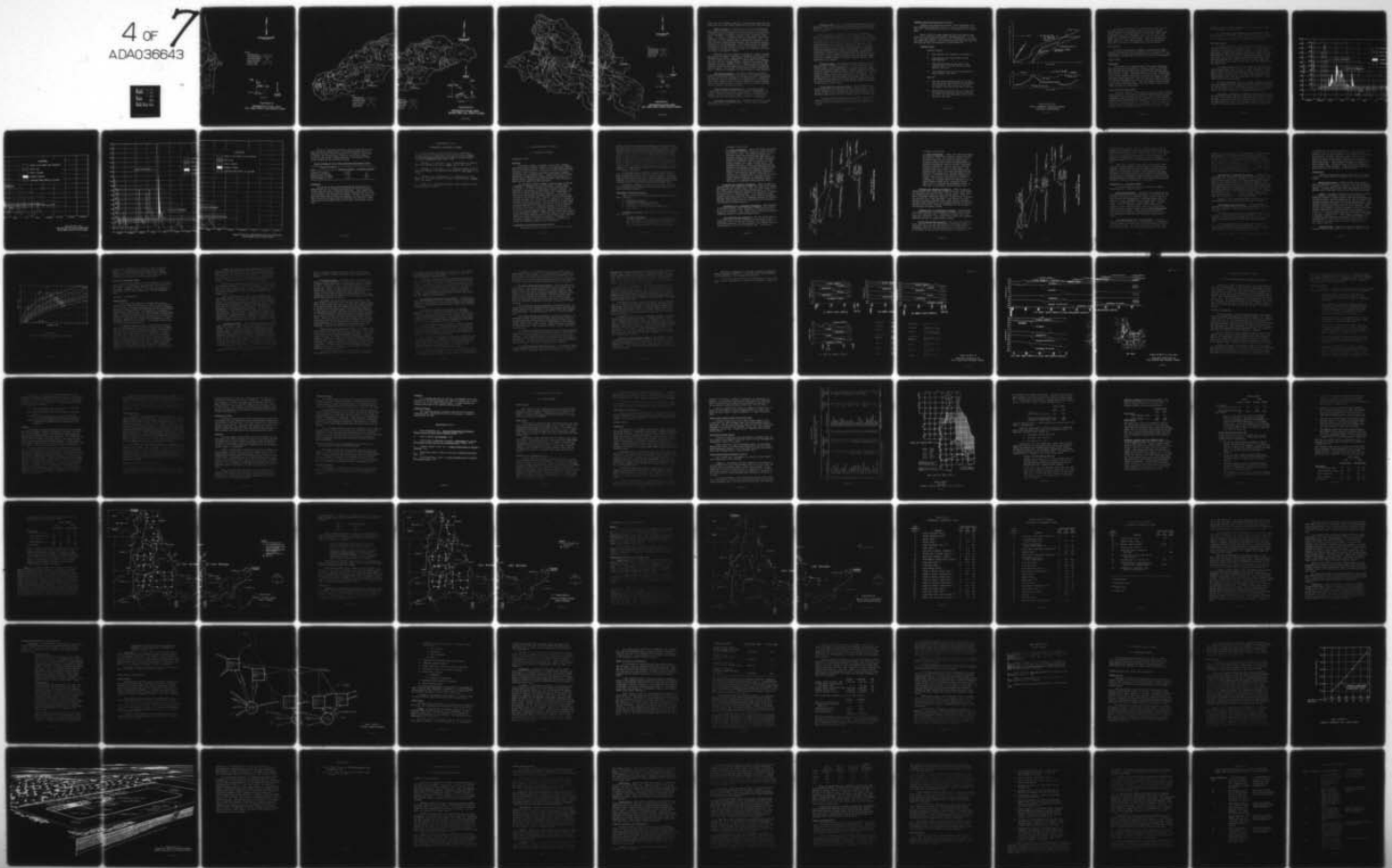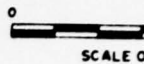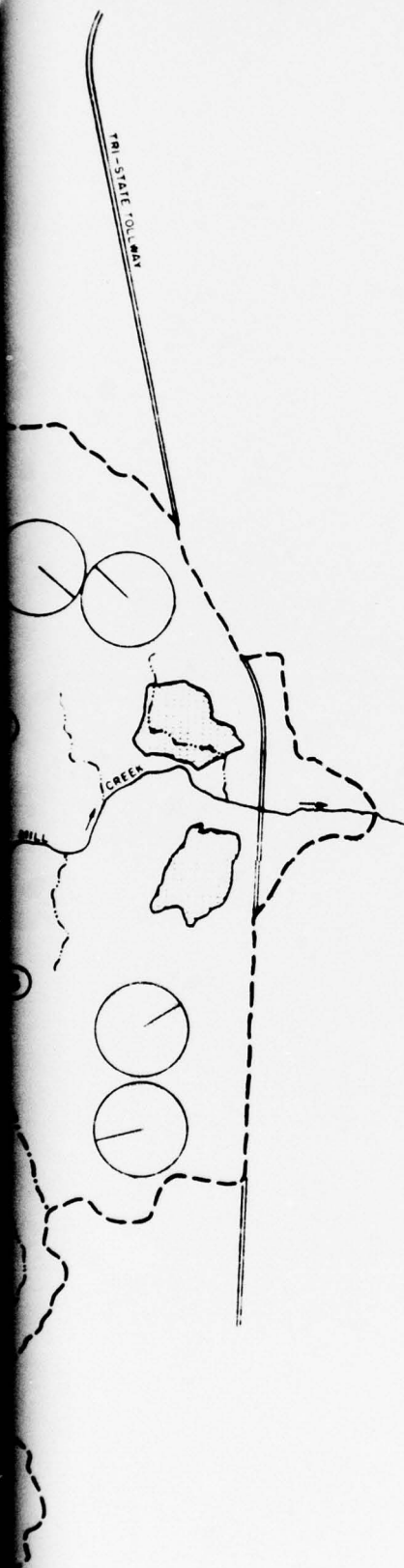
LEGEND:

WATERSHED BOUNDARY   — — — — —

SUB-WATERSHED BOUNDARY   —·—·—·—

SUB-WATERSHED NUMBER   ③

INTERMITTENT STREAMS   – – – – –

PERENNIAL STREAMS   ————

DIRECTION OF FLOW   ——▶   ——▶

Figure B-IV-D-20

REPRESENTATIVE RURAL BASIN
JACKSON CREEK, WILL COUNTY, ILLINOIS
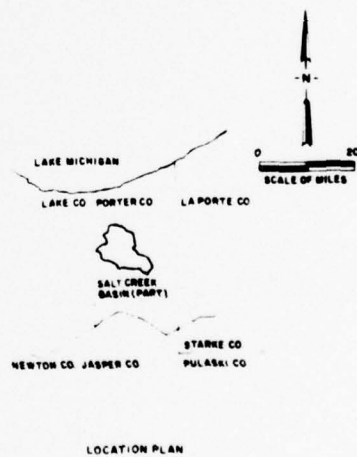
B-IV-D-54

Figure B-IV-D-21

REPRESENTATIVE RURAL BASIN
SALT CREEK, (PART) PORTER COUNTY, INDIANA

B-IV-D-55

feeder or overland grassed waterways. Larger diameter pipes are pro-
vided under interceptor grassed waterways to carry the cumulative flow.

Retention basins. A careful scrutiny of the 22 subwatershed
areas studied indicated that in no case is sufficient relief available
to provide valley-type storage for the required amount of storage with-
out the utilization of inordinate amounts of land as retention basin
areas. In addition, a completely diked or above-ground basin would
produce an unsightly condition and limit the recreational value. For
this reason, a single retention basin configuration, employing a par-
tially excavated, partially diked reservoir, was selected for universal
application in all subwatersheds. Such retention basins are designed
to balance as well as possible all cut and fill operations. Excess
cut material is utilized to shape the sides of the retention basin area
to enhance its recreational function.

Retention basins are designed to provide 2.5 inches of storage.
The need for this amount of storage is established in the section on
Storage Selection above. Working depth of the retention basin is
12.5 feet, with a 2.5 foot freeboard. Overall depth of the retention
basin is 15.0 feet. Spillway design is based on the maximum storm
of record which was recorded in 1951. Detention basin linings are
created by scarifying and compacting the top one-foot of the exposed
reservoir bottom. The quantity of fine-grained material naturally pre-
sent is sufficient to accomplish the desired degree of water-tightness.

Grit removal facilities. Calculations indicate that reduced
sediment loadings expected from the improved land management greatly
decreases the degree of siltation in the retention basin. Loadings for
the retention basin are estimated to be approximately 0.2 inches per
year, which requires a 60-year period to produce a one-foot sediment
depth in the retention basin. For this reason, grit removal facilities
are not included in the basis of design.

Pumping facility to irrigation area. The pumping facility
capacity will vary with the size of the particular subwatershed. As
a basis of design for irrigation pumping stations, a pump-out rate
from the retention basin to the irrigation machine is established as
0.0028 cubic feet per second per gross acreage of subwatershed.

Conveyance to irrigation area. Standard pressure pipe is used
to transfer flows to the irrigation area. Maximum design velocities
are six feet per second.

Irrigation system. The center-pivot spray irrigation machine is designed to reflect all considerations for this item as outlined in the Appendix B, Section IV-A, and as mentioned in Section 1 of the Data Annex B, Section IV.

Specific length of center-pivot spray irrigation laterals is different in different subwatersheds, and is dependent upon available land as reflected through current land use, topography and other design constraints. The area to be irrigated is a function of the total yearly amount of runoff. This figure is typically 12 percent of the gross subwatershed area, based upon an average application rate of 3.5 inches per week over a total active irrigation period of 30 weeks. Hydraulic capacity of any one rig is calculated upon the percentage of the total irrigated area it serves. For example, if a particular subwatershed irrigation area is divided between two equally-sized pivot machines, each machine would be sized to accept 50 per cent of the flow. Design flow rates are based upon the irrigation requirement of 4.5 inches per week as a design maximum; average weekly application is 3.5 inches.

Drainage system. The underdrain system installed to capture renovated flow is designed to receive 3.5 inches per week of in-filtrated water. Plastic drainpipe laterals are envisaged at a depth of five feet below the ground surface. Concrete main pipes collect flows from drainage laterals and convey them by gravity to the nearest perennial stream. Some of the arriving flow is diverted via a piping system to a nearby well and injected into the local aquifer. The aquifer serves as a long-term storage of renovated water which can be pumped-out for potable needs during periods of prolonged drought. The basis of design for such an aquifer storage system is presented in Data Annex B, Section IV-B.

Power supply and operation system. Power requirements for irrigation pumping and center-pivot systems form the basis of design for the capacity of each subwatershed power supply system. This system includes electrical transmission lines and a transformer station. Electrical transmission lines are assumed to be placed underground.

Standard operating equipment associated with a package lift station is envisioned for the irrigation pump facility. Likewise, standard center-pivot irrigation machine operating facilities are en-visioned.

Operation and Maintenance Basis of Design

Operation and maintenance features of most components in the rural stormwater management system are covered in Appendix B, Section VI-D. A brief discussion follows on the operation of the retention basin.

Figure B-IV-D-22 shows generalized inflow and outflow mass curves for a 2,000-acre representative subwatershed basin. In addition, this figure presents a lake level plot corresponding to the inflow and outflow mass curves. The following operating rules are given and also form the basis for information in Figure B-IV-D-22.

Operation Rules

Irrigation Machine

a. Stop operation from November 15 to March 15.

b. Stop operation when large storms saturate irrigation areas.

c. Start operation when the lake level in the reservoir rises to one foot above the design minimum lake level.

d. Stop operation when the lake level reaches the design minimum lake level.

Reservoir

a. The crest of the spillway is set at 12.5 feet from the bottom of the reservoir from September 1st to April 14th and is set at 10 feet from the bottom of the lake for the rest of the year.

b. The minimum lake level is set at 2.5 feet from the bottom from September 1st to April 14th and is set at 5 feet from the bottom of the lake for the rest of the year.
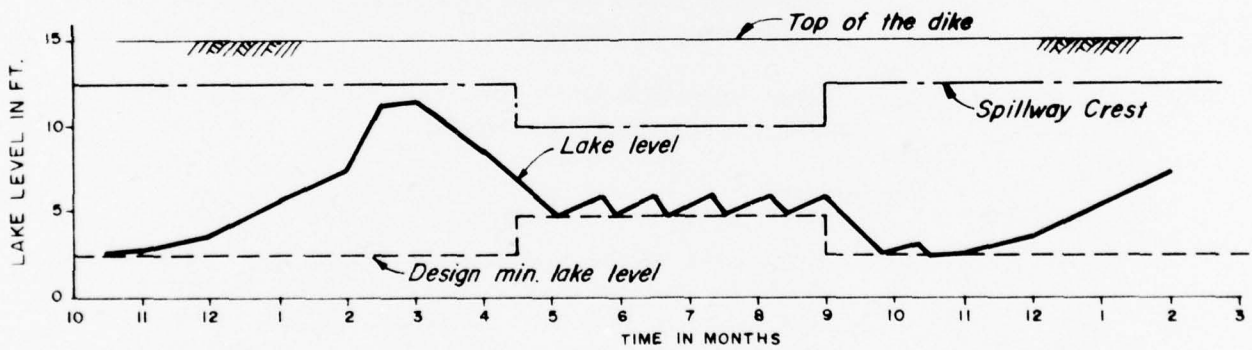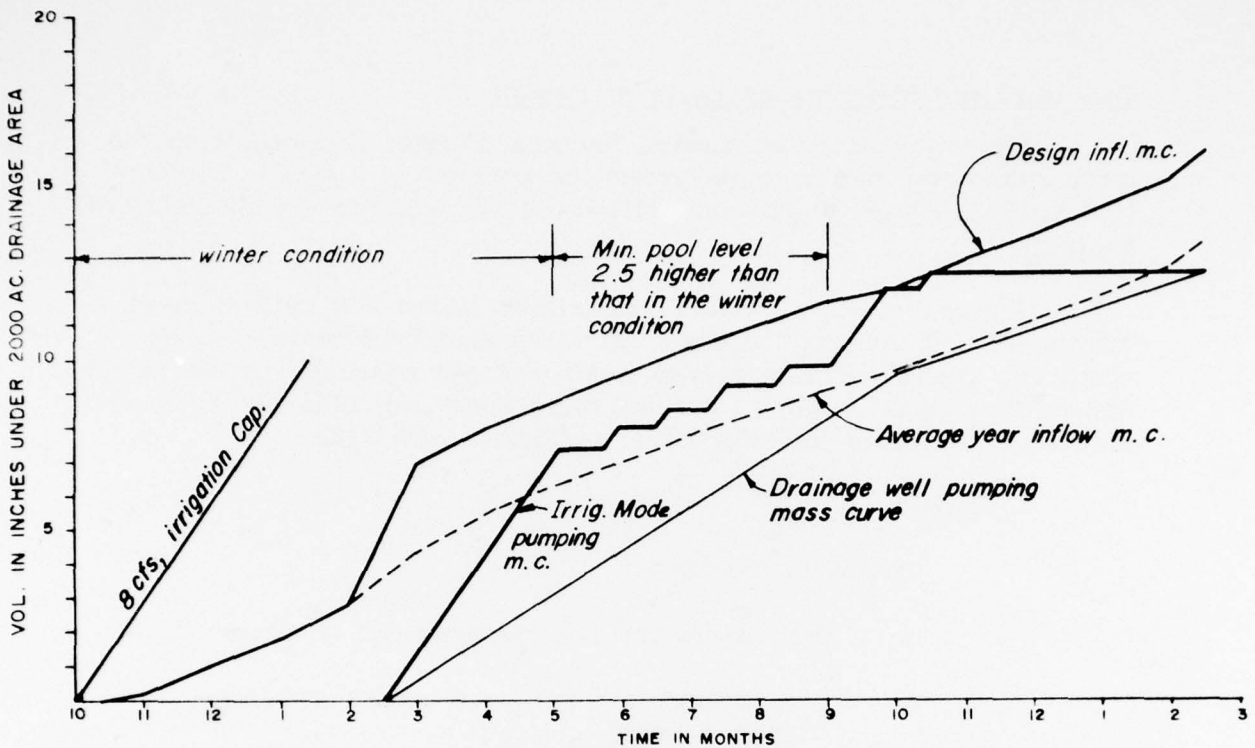
Figure B-IV-D-22
RURAL STORMWATER RETENTION BASINS
HYDROLOGIC PERFORMANCE

B-IV-D-59

Recreational possibilities are directly related to the level of the water surface in the retention basin. Due to the relatively shallow depth of the facility during the summer, body contact sports are probably not a viable recreational use. However, small craft sailing is an attractive and feasible use with the 5-foot minimum depth called for over the summer months. With the permanent pool, winter sports are a very good possibility. Sides of the retention basin are not so steep as to prohibit access for this purpose.

CONCLUSIONS

Collection and delivery to storage of stormwater from urban, suburban and rural runoff will undoubtedly have a beneficial effect or decreasing the amount of flooding occuring in the C-SELM area. Flood relief benefits are discussed in the following section of this Appendix.

FLOOD RELIEF

Introduction

Flood relief is an important aspect of this study because all four major rivers, the Des Plaines, Chicago, Calumet, and DuPage, which run through the C-SELM study area experience frequent overbank flooding. In most cases this flooding is a natural consequence of heavy storm runoffs. Inundation of flood plains is a natural occurrence, having happened historically long before man began to interfere with these natural phenomena. With the increasing pattern of urban and suburban development, however, the unit rate of direct runoff from a heavy storm increases yearly. Thus, the overbank flooding problem should become even more severe in the future unless effective flood-control measures are implemented.

Basic Methods of Flood Control

Two basic methods of flood control have normally been employed in the study area, the creation of stormwater storage reservoirs and the improvement of the existing river channels. Using the latter method necessarily produces adverse effects on downstream areas unless comparable channel improvements are constructed. On the other hand, the flow from a storage reservoir can be regulated so that it causes no adverse effects in any part of the downstream river system. For this reason, the storage reservoir method is used in this study for regula-

lation and control of stormwater quality because it is also of value
in achieving flood control benefits.

As is described in the stormwater management section of the
basis of design, a stormwater storage reservoir has a pump-out rate
of 0.002 and 0.004 cfs per acre and a storage capacity of 2.85 inches
and 2.5 inches for the suburban and urban areas, respectively.

Simulation Analysis

A "dynamic simulation model" using the Des Plaines River as
the typical waterway was created for the purpose of analyzing the per-
formance of the stormwater management system. Using the wettest
year (1951) on record at Des Plaines, Illinois, as a basic example,
hydrographs for existing and design conditions were obtained by flood
route analysis. The results of this analysis are presented in Figure
B-IV-D-23. These results indicate that the peak flow for the river
would be decreased from 2,300 cfs to 950 cfs. This is less than
the estimated bank-full capacity of the river; thus, no overbank flood-
ing would occur.

The largest single storm flow ever recorded at the Des Plaines,
Illinois, gaging station was 3,750 cfs, in 1950. The results of the
above analysis using this flow shown on Figure B-IV-D-24, indicate
that the peak river flow would be reduced to 390 cfs for design con-
ditions versus a peak flow of 3,750 cfs for 1950 conditions. This
great decrease in the flood peak results from the fact that no large
storms preceeded this major storm.

Since the other three rivers, the Calumet, Chicago, and DuPage
have flow patterns similar in nature to those of the Des Plaines River,
the same performance can be expected for these rivers as is predicted
for the Des Plaines River. All of these watersheds can be character-
ized as involving a combination of suburban and rural land use.

The Chicago Underflow Plan adopted in this study for the urban
area is described more completely in the report by the State of Illinois,
et. al.[1] The 0.004 cfs per acre pumping capacity and the 2.5 inches
of storage capacity proposed in this C-SELM study for the urban area
are equivalent to the 1,000 cfs of pumping capacity and the 50,000
acre-feet of storage capacity proposed by the Chicago Underflow Plan.

LEGEND

☐ WATER TO BE PUMPED FOR TREATMENT

▨ BASE FLOW

▨ STATIC STORAGE

■ DYNAMIC STORAGE

▨ OVERFLOW FROM CREST OF SPILLWAY

HYDROGRAPH

JUNE JULY AUGUST SEPTEMBER OCTOBER NOVEMBER DECEMBER
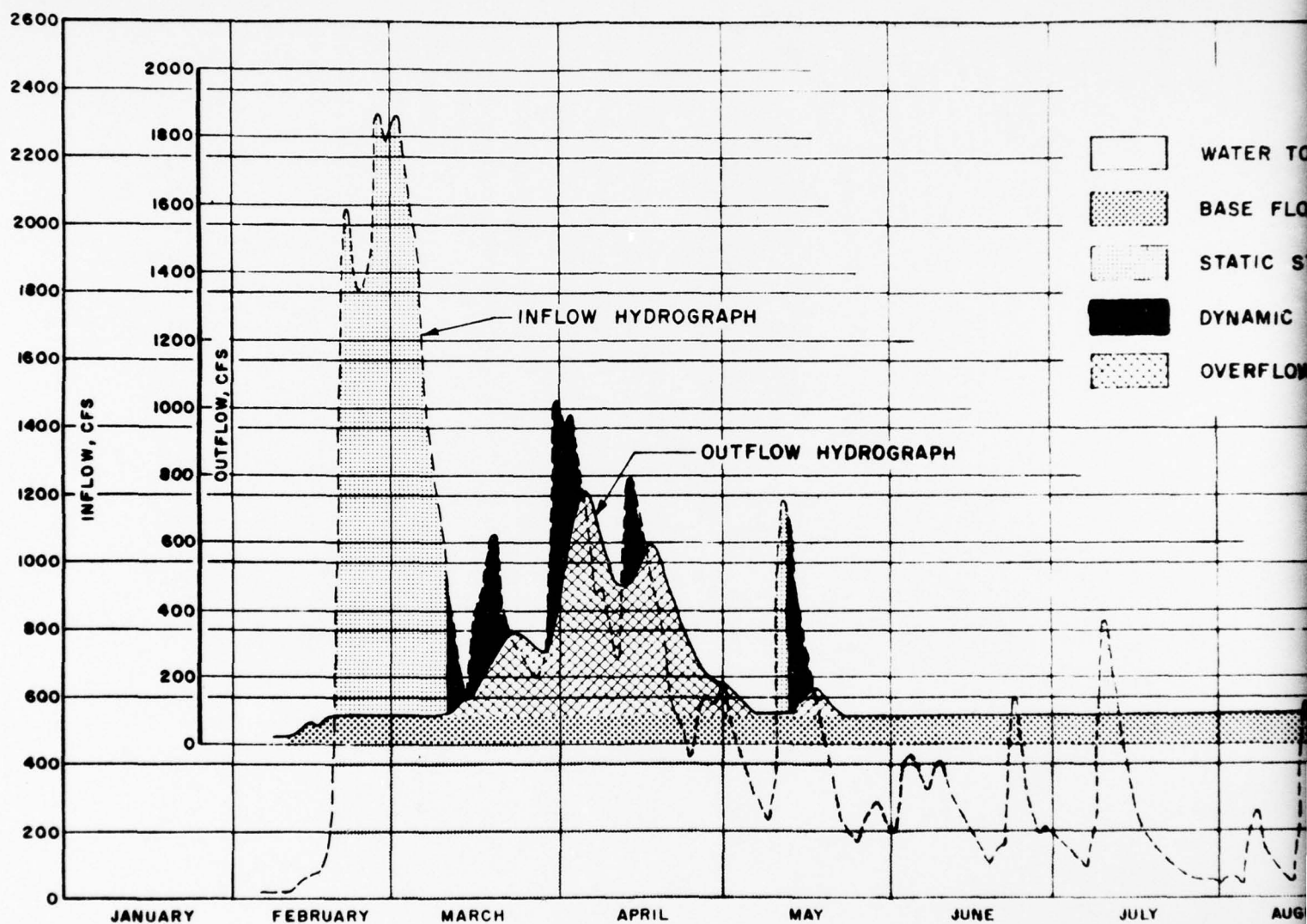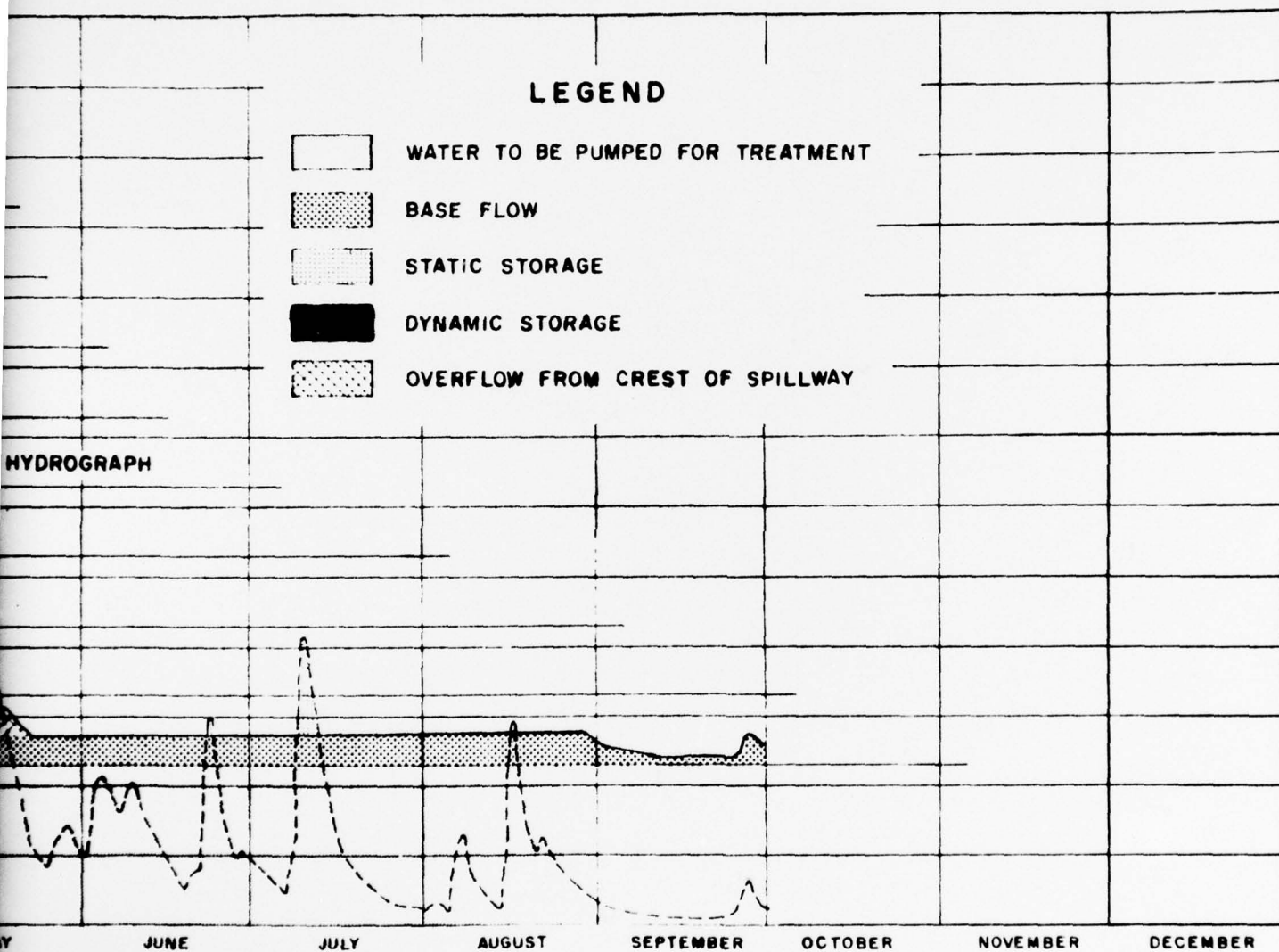
Figure B-IV-D-23
1951 INFLOW—OUTFLOW HYDROGRAPH
DESPLAINES RIVER AT DESPLAINES
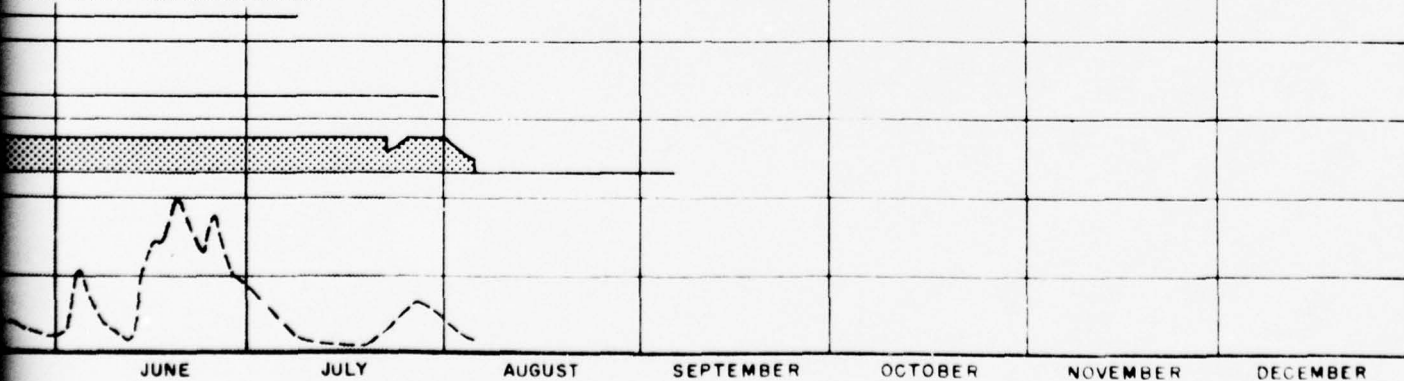
B-IV-D-62

Figure B-Ⅳ-D-24 1950 INFLOW-OUTFLOW HYDROGRAPH
DESPLAINES RIVER AT DESPLAINES

Based on a continuous simulation analysis of the past 21 years using the "Single Storage Model", the Chicago Underflow Plan[1] has determined the total number of overflows in 21 years, the total hours of spillage, the maximum days in storage, and the total overflow volume of water for the C-SELM urban area. The following are the results obtained by this simulation analysis.

System Performance of the Urban Stormwater Management System

Item	Analyzed Condition Existing Condition	Proposed Condition
Number of Overflows	2,014	8
Total hours of spillage	32,104	85
Maximum days in storage	0	122.6
Total Overflow (ac.-ft.)	4,780	168

Conclusion

The results of this analysis indicated that there would be a great reduction in the number of spillages and in the total volume of spillage with this type of stormwater management. Thus, storm flows in the C-SELM study area rivers are greatly decreased with this significant reduction in the spillage of stormwater. It should be noted, also, that these rivers receive stormwater spillages from rural and suburban areas as well as the urban area, and therefore, the reduction in storm flows would be the combined effect of all forms of stormwater management.

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IV. COMPONENT BASIS OF DESIGN

E. CONVEYANCE SYSTEMS

CONVEYANCE TYPES

Definition

Conveyance systems, as defined for this study, transport wastewater flows to treatment facilities. There are several distinct types of conveyance systems. These include stormwater conveyance, municipal and industrial (M & I) conveyance, combined stormwater-M & I conveyance, and a combined conveyance system which transports wastewaters to land treatment sites. Flows handled by all of these systems are conveyed either in pressure pipes by pumpage or in gravity tunnels. Gravity tunnels are used when large quantities of water must be handled over great distances.

All conveyance systems originate at storage sites as defined in Appendix B, Section IV-D, or access points. Access points, by definition, are former treatment plant sites which have been eliminated in the process of regionalizing treatment facilities. The reference or starting point for the C-SELM study acknowledges the plans of the various regional planning agencies. These plans established 64 regionalized treatment plants as the immediate planning goal for the C-SELM area. Presently, there are 132 treatment facilities throughout the C-SELM area. A reduction of the number of treatment facilities from 132 to 64 is accomplished as a prerequisite to C-SELM study considerations. The cost of conveying flows from abandoned facilities to the remaining 64 plants is not included in the scope of this study. Consequent reduction of the number of plants from 64 to any lesser number of regionalized plants creates a need for diverting wastewater flows, formerly entering these plants, to the regionalized facilities. Termination points of all conveyance systems are either treatment plants or land treatment sites, depending upon which treatment technology is employed. Furthermore, all Conveyance systems are regulated flow systems. Regulation as applied to conveyance systems is discussed below.

Unregulated vs. Regulated Conveyance Systems

Unregulated conveyance systems are by definition collection

systems which convey unregulated stormwater and dry weather flows from their points of origin to treatment facilities or storage. A good example of this type of collector is the tunnel system of the Chicago Underflow Plan, described in the Appendix B, Section IV-D. Design criteria for such a system takes into consideration maximum design flows, and provides space for their acceptance within the collector facility. Such a system was investigated and costed out for the entire C-SELM area. It involved many miles of large tunnels which served as collectors for storm overflows and which delivered their flows to centralized storage sites. While this is expedient for the C-SELM urban area, because of the existing combined flow collection systems, it is expensive and cumbersome for the remaining developing suburban C-SELM areas.

On the other hand, a regulated conveyance system starts at either a stormwater storage facility or an access point. Both of these facilities deliver flows to a conveyance system at a regulated pump -out rate and therefore permit a more economical system design, responsive to continuous and uniform flow characteristics rather than to peaking needs to accommodate unregulated stormwater flows.

In addition, regulation of flows into the treatment facilities permits more equalized operation of these facilities and makes them more economical to operate.

Description of Conveyance Systems

There are four types of conveyance systems employed in this study. They are:

    a. Stormwater conveyance
    b. Municipal and Industrial conveyance
    c. Combined M & I-stormwater conveyance
    d. Land treatment site conveyance, a special case
       of c. above.

Stormwater Conveyance. This system consists of the following two subsystems:

    1. Basic Conveyance
    The basic stormwater conveyance system transports flows from mined storage or shallow pits to basic treatment plant facilities or basic access points. There are 36 such basic treatment plant (or access point) facilities

considered in this study.

2. Future development. Future (1970-1990) development will require a stormwater conveyance system from suburbanizing rural areas. The system envisioned conveys stormwater flows from previously-described storage facilities provided either through the implemented rural stormwater management system or through a mechanism provided by developers, who may construct localized storage facilities as a matter of public policy. The developers will have a choice of either delivering the regulated flows directly to outlying stormwater retention basins or to unregulated-flow, grassed waterways constructed as part of the rural stormwater management system. The stormwater retention basins will then be connected by a pumping station and pressure line to the nearest treatment plant site or conveyance system. Figure B-IV-E-1 illustrates the distribution network of both types of conveyance system discussed above.

Municipal and Industrial Conveyance. There are some 28 treatment plant sites, within the C-SELM study area, which handle only municipal and industrial flows, both as a result of the character of their service areas and optimal design policy. There is, also, an infiltrated stormwater component in these flows, accounted for within the M & I flow framework. Upon adoption of centralization for treatment facilities, these plants would be abandoned and their flows transferred to the centralized facilities for treatment. Transfer of these flows would be accomplished via the municipal and industrial conveyance system.

Combined M & I - Stormwater Conveyance. Regionalization of treatment facilities, which eliminates treatment plants handling M & I and stormwater flows, contributes to creation of the combined flow conveyance. This system conveys flows from eliminated treatment facilities to regional treatment plants.

Land Treatment Site Conveyance. The last of the systems to be discussed is the land treatment site conveyance system. In alternatives which consider land treatment as a viable means of wastewater renovation, all flow generated in the C-SELM area must be transported to several land treatment sites. The means of

Figure B-IV-E-I
SCHEMATIC CONVEYANCE SYSTEMS

considered in this study.

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Land Treatment Site Conveyance. The last of the systems to be discussed is the land treatment site conveyance system. In alternatives which consider land treatment as a viable means of wastewater renovation, all flow generated in the C-SELM area must be transported to several land treatment sites. The means of

Figure B-IV-E-1
SCHEMATIC CONVEYANCE SYSTEMS

transporting these flows is a system of large tunnels collecting wastewater at regionalized access points and conveying it to the respective land treatment areas. The diameter of the tunnels varies from 10' to 23'; the tunnels are located up to 700 feet below the surface in the Silurian Dolomite system--Hunton Megagroup, or Ordovician Dolomite System -- Galena and Platteville groups. Schematic Figure B-IV-E-1 shows different types of conveyance systems.

Alignments for tunnel routing follow existing rights-of-way along major highway corridors to avoid need for involving private interests. Access shafts, as well as ventilation shafts along the length on tunnel runs are provided to ensure proper access and ventilation. Distribution tunnels at land sites deliver wastewater to pumping stations which lift it from the tunnels to treatment and storage lagoons. Distribution tunnels at treatment sites as well as wastewater pumping stations are part of the land treatment technology and are discussed in Appendix B, Section IV-A.

Design Basis of Conveyance Systems

The Design Basis for each of the conveyance systems described above is discussed below.

Stormwater Conveyance. Conveyance lines leading from the stormwater storage sites to treatment plants, or access points, are designed on the basis of a pump-out rate of 0.002 cfs per acre for a drainage area contributory to a particular storage site. For example: assume a drainage area of 6 square miles, or 3,840 acres. The conveyance line is then designed for Q = 0.002 x 3,840 = 7.68 cfs. Average velocity of flow in the line is assumed at 6 fps (feet per second). The size of line and the friction head loss is then selected from the Pipe Friction Manual of the Hydraulic Institute. With a known length of pipeline, total head loss is computed. Pumping station selection can be made depending on total head loss and other assumed friction losses.

Municipal and Industrial Flow Conveyance. Conveyance lines leading from existing M & I treatment plants to regionalized treatment plants, or access points, are designed on the basis of the following factors: (a) average daily plant dry-weather flow, (b) peak diurnal flow, and (c) stormwater infiltration. The plant

capacity is equal to the sum of the average daily dry-weather flow with infiltration. If the peak diurnal flow exceeds the above-mentioned capacity, then storage is provided at the plant site to regulate these peaks. For the description of storage so provided, refer to Appendix B, Section IV-A. For the description of treatment plant capacities, refer to Appendix B, Section IV-A. In conclusion, therefore, the conveyance lines are designed on the basis of plant average daily dry-weather flow with stormwater infiltration. Selection of line and pumping station sizes follows guidelines described in the stormwater flow conveyance discussion.

Combined M & I Stormwater Conveyance. Combined flow conveyance lines leading from existing treatment plants which handle dry-weather M & I as well as stormwater flows are designed on the basis of the following factors: (a) average daily plant dry-weather flow, (b) peak diurnal flow, and (c) stormwater infiltration, and (d) stormwater pump-out rate from localized stormwater storage sites. If the peak diurnal flow exceeds the average daily plant dry-weather flow with infiltration, plus stormwater pump-out rate, then storage as described for M & I flow conveyance is provided. In conclusion, the conveyance lines are designed on the basis of plant average daily dry-weather flow with stormwater infiltration and stormwater pump-out rate flows from contributing storage sites.

Selection of line and pumping station sizes follows guidelines described in the stormwater flow conveyance discussion.

Land Treatment Site Conveyance. Design parameters for wastewater conveyance tunnels are based on experience gained by the City of Chicago in connection with studies on the Chicago Underflow Plan.

The capacity of the tunnels is designed to accommodate the summation of all contributing treatment plant flows. These flows include (1) average plant dry-weather flows with infiltration and (2) design stormwater pump-out rate flows from contributing storage sites.

A design cross-section of the tunnels is assumed to be circular for simplification of drilling operations. Use of mole-boring machines produces smooth walls which permit use of Manning coefficient of $n = 0.017$ in the design formulas.

A minimum velocity of 2.0 feet per second is used to assure

no settling of solids during transmission. Tunnel slopes vary from 0.0005 to 0.001 to insure that this minimum velocity will be exceeded under most flow conditions. Plots showing the elevations of the geologic formations along the routes of the tunnels were produced to obtain proper vertical positioning of the tunnels. General description of the geology of the region is enclosed at the end of this section. Typical geologic profiles of the region are also presented here. Curves for sizing unlined mole tunnels are used to obtain tunnel diameters. These curves were developed for the present study and are shown on Figure B-IV-E-2.

Specifications

Following guidelines were used in selection of force mains, tunnels and pumping stations constituting part of the conveyance system.

Pressure Force Mains. Average velocity for all pressure lines was selected to be 6 fps. However, with limiting maximum size of line at 84", velocities as high as 15 fps were occasionally reached. Concrete pressure force mains were selected as the most probable type of pipe which would be used throughout the system.

Tunnels. Conveyance tunnels are, for the most part, located in the sound rock formations of Silurian Dolomite or Galena-Platteville Dolomite. Tunnels drilled in this formation generally do not require lining. This was verified by the City of Chicago in the recently constructed tunnels for the Chicago Underflow Plan. Construction of tunnels is based on the use of mining machines (moles) capable of drilling tunnel sizes up to 35 feet in diameter. There are many fault systems in the rock formations in the C-SELM region. Further investigations may require changes in the alignment of tunnels. It is also envisioned that some tunnel reaches may have to be lined. Excavated rock material and its disposal is discussed in Appendix B, Section IV- F . The geologic considerations used in selecting proper geological formations for mole tunneling are discussed below. Also included are some typical profiles of the geological formations which are found in different parts of the C-SELM study area.

Pumping Stations. Proper and continuous operation of all pumping stations in the system is the singlemost important factor
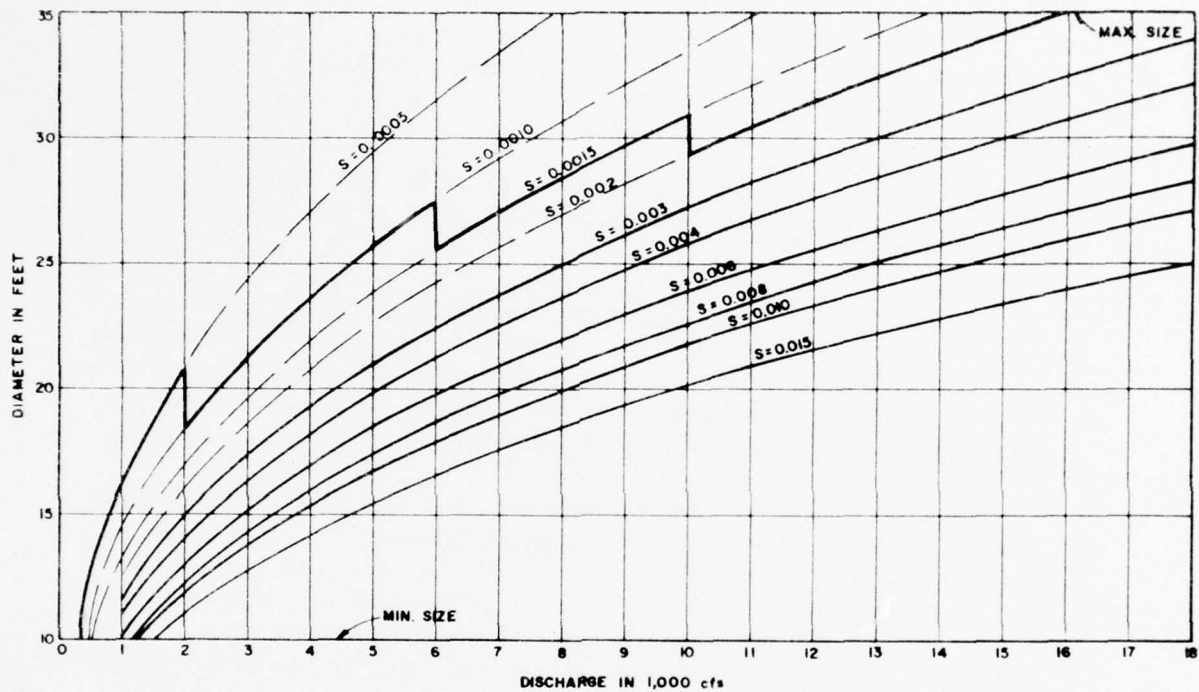
Figure B-IV-E-2

CURVES FOR SIZING OF UNLINED MOLE TUNNELS
(n = 0.017)

in the proper functioning of the entire wastewater management
system. It is therefore part of the design that the pumping
stations are equipped with emergency power sources capable of
entering service at a moment's notice. Spare pumps are also
included as an integral part of any pumping station.

Life of the Conveyance System

All systems are designed for a life expectancy of 50 years.
Force mains and tunnels do not have replacement factors built into
the system. Pumping stations, however, have parts replaced
on a regular replacement schedule. Some parts are scheduled for
replacement every 25 years, others are on 10-year replacement
basis.

GEOLOGIC CONSIDERATIONS

General Geology

During the geologic epochs from Middle Ordovician to
Devonian, the lower Lake Michigan area generally was beneath an
arm of the sea. This marine environment was suitable for sedimen-
tation and successive strata were deposited until the total thickness
reached several thousand feet. The formations are composed of
several different types of rock and of many gradations between these
types. The different lithologies are important factors affecting the
suitability of these strata for rapid tunneling operations. Similarly,
structural changes that have occurred since deposition have had a
significant effect on the rock quality. Such changes include fault-
ing, folding and localized erosion.

An especially important factor in the formation of this rock
was the changing position of shorelines during the time of deposi-
tion. This resulted in the introduction of clastic sediment ranging
from sand to clay, during the calcareous deposition. A disad-
vantage of this clastic deposition is the existence of numerous
thin clay seams. Such a clay seam is likely to be an annoying
source of water, especially where a dip of the strata coincides
with the grade of the tunnel so that a single seam may be exposed
along a considerable length of that tunnel. Another construction
problem could occur where the joint spacing becomes small with
relation to the diameter of the bore. Under such a condition, a
clay seam just above the tunnel roof could permit blocks of rock to
drop.

Probably there was further marine deposition during the three geologic periods that followed the Devonian, but since that time there has been 250 million years of emergence during which erosion and weathering have been the dominant processes. The record of these three periods is inferred almost entirely from other locations because the post-Devonian rocks have been removed from this area as well as most of the Devonian and some of the Silurian.

This extensive erosion has had significant effect on some of the engineering properties of the Silurian formations. Stress release from the removal of many hundreds of feet of overburden has opened joints in the Upper Silurian strata. Exposure to weathering from the surface has widened such joints and started solution along bedding planes.

About a million years ago, there came an abrupt change. Continental glaciation brought a series of ice advances across the site. Most of these ice sheets had a thickness in the thousands of feet, so the pressures available for erosion and the quantities of water available for transport of sediment were quite large. Carried away were the residual soils and the rock that had been partly decomposed by the long period of weathering. Scraped off and removed was a considerable thickness of rock, as in the basin of Lake Michigan. The rock surface so exposed was unusually clean and sound for a natural formation. Deposition by the glacier was of two kinds. Glacial till is an unsorted mixture of soil and rock particles ranging in size from clay to boulders. Outwash or valley train is a granular deposit that was laid down by glacial meltwater.

The Devonian System. Rocks of the Devonian age commonly overlie the Silurian in Indiana with a marked erosional unconformity, although the boundary often is difficult to distinguish in the subsurface. Such an unconformity indicates that the lower formation was subjected to a considerable period of weathering and erosion before the upper formation was deposited. These strata consist primarily of carbonate rocks, thin shales and occasional evaporite beds. A sandy dolomite may mark the base and these are commonly overlain by a few feet of brown to black shale. In the study area, the Devonian reaches 100 feet in thickness but more commonly it has been eroded completely. The stratigraphy is complex and no formal names have been assigned to the Devonian in this report.

The intermittent nature of the Devonian remnants makes it undesirable for tunneling. The unconformity at its base is the

cause of Devonian fillings in erosional valleys in the Silurian. This is a hazard to tunnels being driven in the upper part of the Silurian.

The Silurian System. Throughout most of the area, the Silurian consists predominantly of light gray to buff or brown, thin to medium bedded, very fine to medium-grained dolomite, calcitic dolomite, or limestone. Gray to green shaly partings are fairly common and, locally, the carbonates become somewhat silty or argillaceous. Chert is commonly present in moderate amounts but may be locally abundant, particularly in the basal portion. A vertical or nearly vertical system of joints is known to occur and, occasionally, solution has widened these joints and they have later become filled with clay or shale.

Of special note in the Silurian is the occurrence, in the upper portion, of mound-like coral and algal reefs. Some of them, such as the Thornton Reef, grew quite large and attained thicknesses of several hundred feet and diameters of several thousand feet. In general, the reefs consist of three major lithologic divisions: (1) the reef core, although bedded, is generally a rather massive or structureless-appearing, silty dolomite or limestone of low porosity; (2) the reef flank is derived from calcareous organic remains which lived on the flank and were broken from the reef by wave action. These are generally rather thinly bedded, porous dolomite or dolomitic limestone and are frequently developed at high angles to the horizontal; (3) an interreef facies, commonly consisting of argillaceous dolomites and limestones, calcareous shales, and siltstones, occurs in the broad areas between the reefs.

Silurian rocks occur throughout the area, but commonly overlie the Maquoketa Group with erosional unconformity. This occurred as a result of structural uplift and subsequent erosion on the crest of the Kankakee Arch following Maquoketa deposition. Throughout much of the area, the Silurian is overlain by Pleistocene glacial deposits. However, in Indiana it is commonly overlain by deposits of the Devonian age. In all cases, the Silurian is bounded above and below by erosional contacts and consequently is quite variable in thickness. It ranges from about 50 to over 400 feet with an average of about 350. In Indiana, it tends to thicken northward toward the Michigan geologic basin.

In practical terminology, the name Niagarian has frequently been applied to the entire Silurian sequence although technically

this applies only to the upper portion of the section. The Silurian of this report consists of the Edgewood, Kankakee, Joliet, and Racine formations in ascending order.

The Silurian rocks lie immediately below glacial deposits in most of the Chicago area so their tunneling characteristics are well known. In the upper part, joints are numerous and probably water-bearing. Horizontal clay seams may produce troublesome amounts of groundwater. The presence of reef structures and of chert concentrations, especially in the upper part, results in lithologic variation that is troublesome for modern tunneling techniques. Similarly, joint concentrations and erosional valleys in the bedrock are most frequent in the upper part.

The Maquoketa Group (Ordovician System). The Maquoketa Group consists primarily of green or greenish gray to brown dolomitic shales with some light gray to brown silty dolomites and limestones. Typically, a well-defined dolomite formation occurs near the middle of the group.

The Maquoketa consists of the Scales, Fort Atkinson, Brainard and Neda Formations in ascending order. The Scales Formation consists of gray to brown, weak to brittle, silty dolomitic shale interbedded with thin layers of fine-grained silty dolomite. In eastern Cook County the entire unit may consist of weak, gray dolomitic shale. The Scales overlies the Galena Group unconformably but is rather consistently 90 to 100 feet thick.

The Fort Atkinson dolomite conformably overlies the Scales and is gradational with it. It consists primarily of light gray to brown, fine to coarse-grained dolomitic shale. The Fort Atkinson is rather consistently present, but it may vary in thickness from 5 to 50 feet. It is formed on a resistant bench where pre-Silurian erosion has removed the overlying Brainard.

The Brainard shale conformably overlies the Fort Atkinson and its gradational with it. It consists of greenish gray, weak, silty, dolomitic shale interbedded with varying amounts of greenish gray, silty dolomite and occasional limestone. The Brainard attains a maximum thickness of around 100 feet, but is thin or absent throughout much of Cook and northern Will Counties.

The Neda Formation appears to be gradational with the under-lying Brainard. It is composed of weak red shale with interbedded pink or green dolomite. Small iron oxide pellets are locally abundant.

The Maquoketa is considered to be a good cap rock for gas storage purposes so it should be an effective barrier to vertical movement of water. This could be a useful property in dewatering a local area during tunneling in the formations below or above it. Some of the shales are not strong enough and the group could not be considered as massive enough for successful tunneling with present techniques.

The Galena Group (Ordovician System). The Galena Group consists chiefly of buff colored, medium-grained dolomite grading into calcitic dolomites with interbedded limestones to the southwest. The Galena averages 200 feet in thickness and normally varies less than 20 feet. Its contact with the underlying Platteville apparently is erosional but is not significantly irregular. Although some thin shaly zones and shale partings do occur, they are not abundant and the Galena dolomites and limestones are relatively pure. The Galena consits of the Gutenberg, Dunleith and Wise Lake Formations in ascending order.

The Gutenberg is the basal formation of the Galena Group. It consits of gray to buff, medium-grained dolomite grading into very fine-grained limestone to the southwest. Thin reddish-brown shale partings are characteristic. The Gutenberg ranges from 5 to 15 feet in thickness.

The Dunleith Formation gradationally overlies the Gutenberg. In northeastern Illinois the Dunleith and Wise Lake Formations often are so similar that they cannot readily be differentiated and they are considered together herein. They consist of light brown to buff, medium-grained dolomite and calcitic dolomite. Widely scattered chert nodules are found throughout. Their composite thickness is rather consistently about 190 feet.

The Galena and Platteville Groups have been tunneled extensively in the lead fields but not by rapid tunneling equipment. These two formations have similar properties and seem to be best suited for rapid tunneling. Joints and bedding planes are narrower and more widely-spaced than in the Silurian and the rock mass should be less permeable. The lithology is uniform and clay seams and chert nodules are not especially numerous. The formation thickness also is uniform.

The Platteville Group (Ordovician System). The Platteville Group consists primarily of gray to brown dolomite and calcitic dolomite interbedded with very fine-grained limestone. The Platteville is present

throughout the area and ranges from about 100 to 150 feet in thickness. In some areas, because of difficulty in placing the contacts, it is grouped together with the Galena both on geologic maps and on boring logs. The contact with the overlying Galena Group appears to be erosional since locally the upper Platteville, Nachusa, may be thin or absent. In ascending order, the Platteville consists of the Pecatonica, Mifflin, Grand Detour, and Nachusa Formations.

The Pecatonica Dolomite is a gray to brown, fine-grained dolomite grading into calcitic dolomite or very fine-grained limestone in DuPage and western Will Counties. It contains thin brown shaly partings and is sandy in the lower few feet. Its contact with the underlying Ancell Group (Glenwood or St. Peter Formations) is erosional but generally fairly sharp. The Pecatonica is present throughout the area and varies from 20 to 50 feet in thickness, being thickest in the southeastern portion of the area.

The Mifflin Formation overlies the Pecatonica and is transitional with it. It consists of light gray to brownish gray, fine-grained dolomite grading into calcitic dolomite and very fine-grained limestone in DuPage and western Will Counties. It contains green to brown shale partings and locally grades into shaly dolomite. Chert is present but rare. The Mifflin is present throughout the area and varies from 20 to 50 feet in thickness.

Throughout most of the area, the Nachusa Formation overlies the Grand Detour with gradational contact. It consists of gray to brown, fine to medium-grained dolomite grading into limestone to the southwest. In general, the Nachusa is the purest and most massively bedded formation in the Platteville Group. It does, however, contain moderate amounts of chert in the lower portion and chert becomes fairly common near the top. The Nachusa is present throughout most of the area, but locally it may be thin or absent. Its thickness is more variable than that of the other Platteville formations, ranging from 1 to 50 feet, and averaging about 20 feet.

The Ancell Group (Ordovician System). The Platteville Group is underlain by the Ancell Group consisting of the St. Peter sandstone and Glenwood Formations in ascending order. The Ancell Group is predominately sandstone but there is some shale and sandy dolomite in the Glenwood. The contact with the overlying Platteville is sharp and erosional in nature.

The group is significant as the base of practical tunneling by methods presently available. It is felt that these sandy formations are too permeable to be considered for tunneling.

The geologic sequence and the structural trends in the C-SELM study area are illustrated in the geologic cross-sections, Figure B-IV-E-3.

C-C NORTH-SOUTH PROFILE

ELEVATION IN FEET

800
700 — GLACIAL DEPOSITS
600 — SILURIAN
500 — MAQUOKETA
400
300 — GALENA
200 — PLATTEVILLE
100
0 — GLENWOOD-ST. PETER
-100
-200

42°00' LONGITUDE T-41-N / T-40-N
T-40-N / T-49 N
T-39-N / T-38-N
41°45' LONGITUDE
T-38-N / T-37-N

DU PAGE CO. | WILL CO.

D-D NORTH - SOUTH

ELEVATION IN FEET

800
700 — GLACIAL DEPOS
600
500 — MAQUOKETA
400
300
200
100 — PLATTEVILLE
0
-100
-200

42°00' LONGITUDE T-41-N / T-40-N
T-40-N / T-39-N

E-E PROFILE IN WILL COUNTY

ELEVATION IN FEET

800
700 — GLACIAL DEPOSITS
600
500 — SILURIAN
400 — MAQUOKETA
300
200 — GALENA
100
0 — PLATTEVILLE
-100 — GLENWOOD-ST. PETER
-200

88°00' LONGITUDE
T-37-N / T-36-N
T-36-N / T-35-N
41°30' LONGITUDE

COOK CO. | WILL CO.

QUATERNARY SYSTEM — Glacial gravels.

DEVONIAN SYSTEM — Undiffe limesto

SILURIAN SYSTEM — Gray to dolomit stone.

ORDOVICIAN SYSTEM

Maquoketa Group — Green light gr limesto the mid

Galena Group — Buff col and cal bedded

Platteville Group — Gray to and cal fine gra Sandy

Ancell Group — Glenwo Fine to shaley

D-D NORTH-SOUTH PROFILE
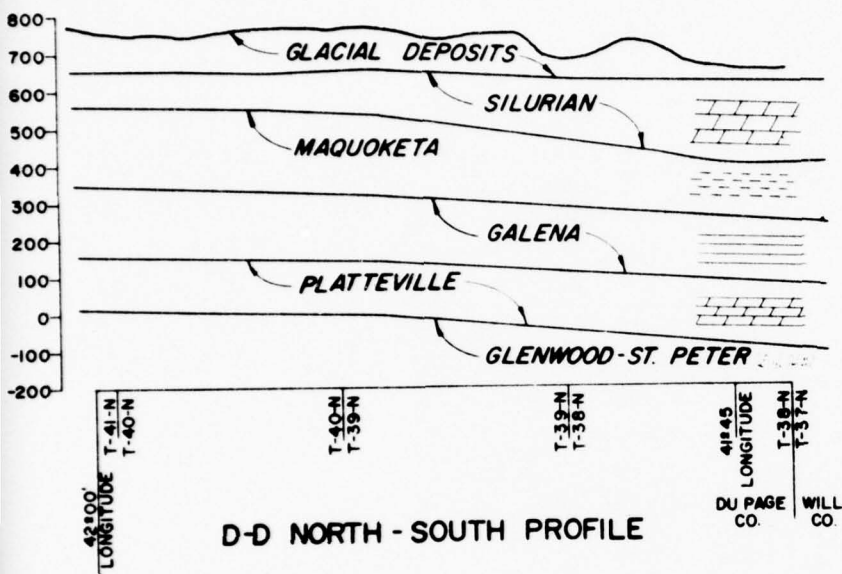
QUATERNARY SYSTEM — Glacial tills and outwash sands and gravels.

DEVONIAN SYSTEM — Undifferentiated light to dark gray limestone and gray to black shales.

SILURIAN SYSTEM — Gray to buff dolomite and calcitic dolomite with some interbedded limestone. Becomes cherty near the base.

ORDOVICIAN SYSTEM

Maquoketa Group — Green to brown dolomitic shale and light gray to brown silty dolomite and limestone. Distinct dolomite unit near the middle.

Galena Group — Buff colored medium grained dolomite and calcitic dolomite with some interbedded limestone.

Platteville Group — Gray to brown fine grained dolomite and calcitic dolomite with some very fine grained interbedded limestone. Sandy at base.

Ancell Group — Glenwood and St. Peter formations. Fine to medium grained sandstone, shaley and dolomitic at top.

Figure B-IV-E-3

GEOLOGIC PROFILES OF
THE C-SELM AND NEARBY AREAS

B-IV-E-16
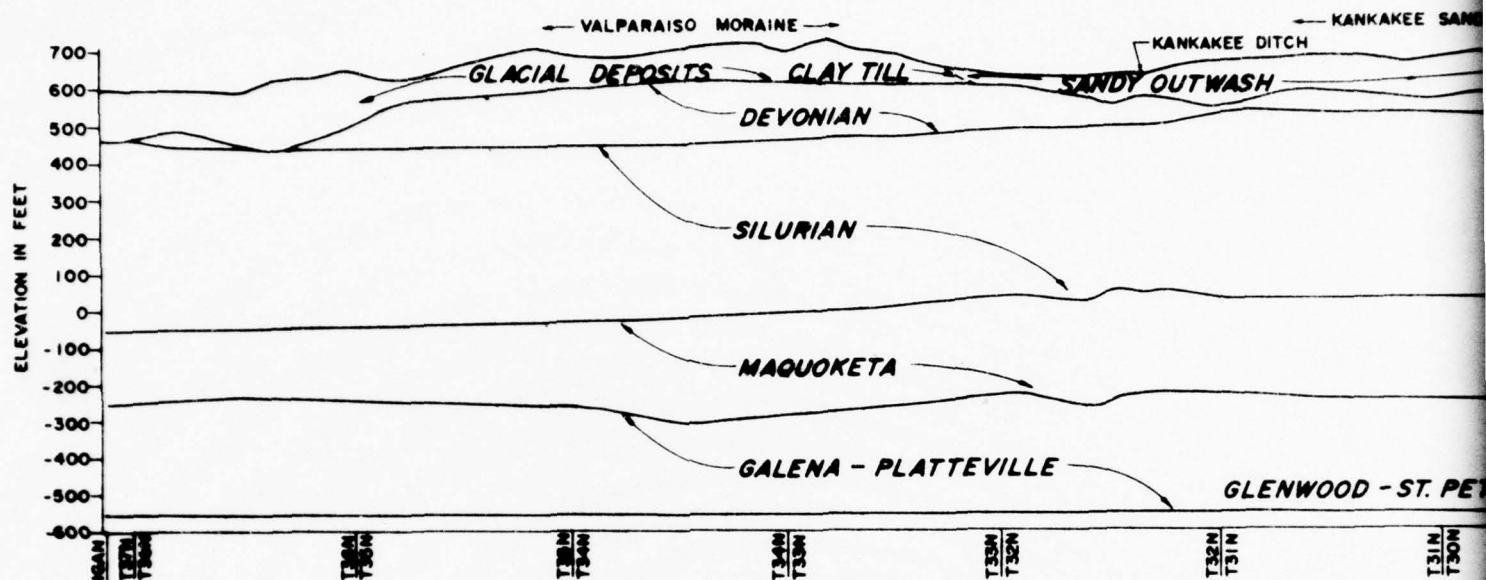
A-A NORTH-SOUTH PROFILE IN LAKE AND NEWTON COUNTIES
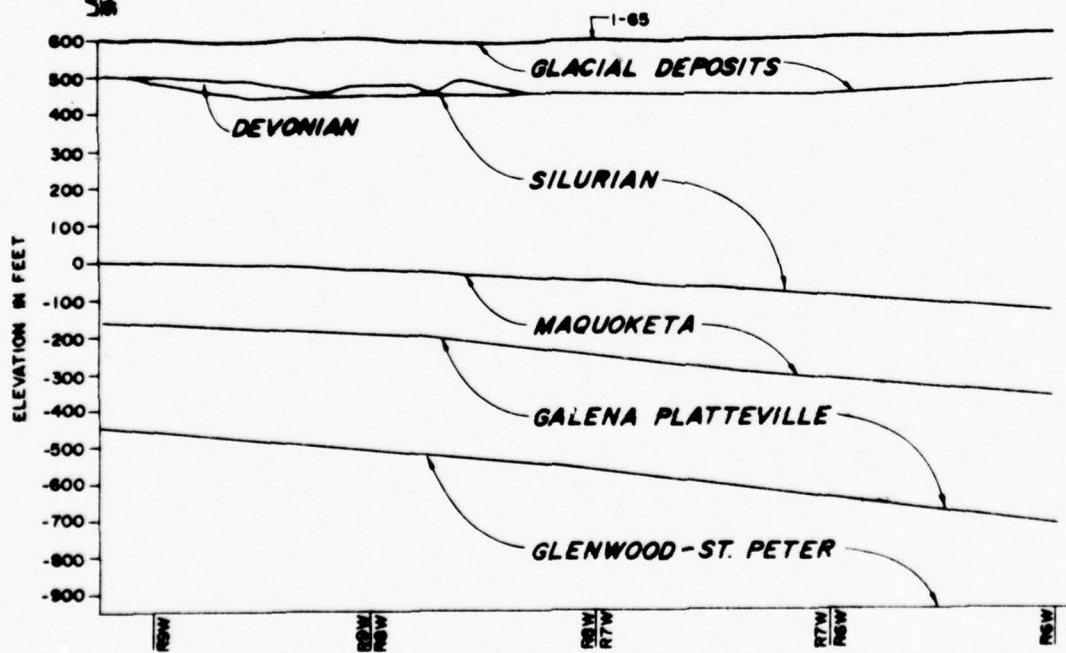
B-B EAST-WEST PROFILE IN LAKE AND PORTER COUNTIES

KANKAKEE DITCH ←— KANKAKEE SAND PLAIN —→ ←— IROQUOIS MORAINE —→

NDY OUTWASH — CLAY TILL

GLENWOOD - ST. PETER

AND NEWTON COUNTIES
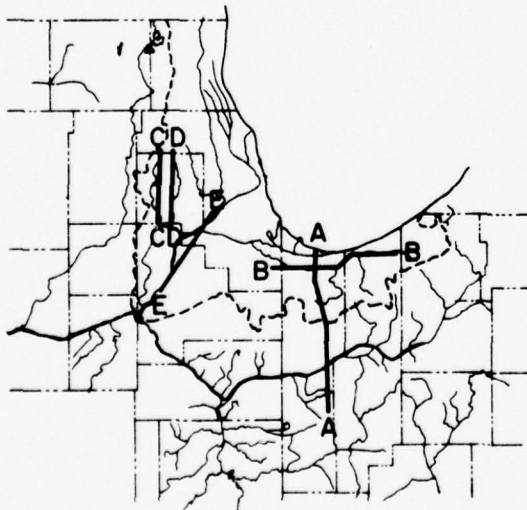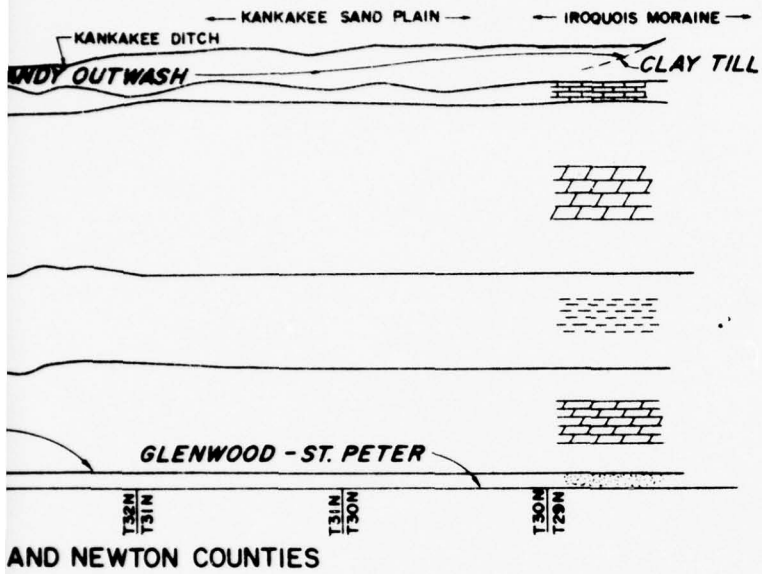
KEY MAP

Figure B-Ⅳ-E-3 (Continued)

GEOLOGIC PROFILES OF
THE C-SELM AND NEARBY AREAS

IV. COMPONENT BASIS OF DESIGN

F. ROCK AND RESIDUAL SOIL MANAGEMENT SYSTEMS

There are a variety of possible options for the management of the rock and residual soil, as was mentioned in Appendix B, Section II-F. The basis of design of three of the systems is described here and the basis of cost for the same three systems is described in Appendix B, Section VI-F. Each system has a major component which encompasses the management of rock from the McCook-Summit storage basin. Other smaller components are utilized for the materials from other sources. The major components of the systems are: (1) a mountain landscape; (2) recreational islands in Lake Michigan; and (3) storage of one-half of the rock for commercial use. Portions of moled rock and soil from the urban area are also managed in the major component.

MOUNTAIN LANDSCAPE

The desirability of a major mountain landscape for recreational use in the Chicago Metropolitan area has been noted. [1] [2] Several possible sites for such a mountain have been located in the southwest Cook County area. [2] The basis of design for such a mountain includes provision for using all of the mined rock and residual rock from the McCook-Summit storage basin plus two-thirds of the moled rock from the urban area. These quantities are nearly the same for each of the wastewater management system alternatives; thus, the mountain design does not depend upon which alternative is selected.

It is not practical to move materials for great distances to the mountain site if there are suitable disposal areas nearby and if they will not add significantly to the recreational value of the mountain. Thus, it is assumed that overburden and rock in the rural and suburban areas is used for smaller sculptured landscapes, less than 100 feet in height, within a reasonable transport distance from the source.

The basis of design of the mountain landscape also assumes that one-third of the moled rock in the urban areas is retained within the area from which it originates. This rock would be provided, at the discretion of the local community, for use as a landscaping resource. The extent of the tunnel system and the variety of potential uses near

the various outlet shafts could make this use of material very attractive. Each community would have the choice of accepting or rejecting the materials originating from it. Sculpturing of open space within the urban area could add attractive recreational facilities. Infilling of abandoned pits is another possible use.

Estimated Quantities

The quantities of materials requiring management from the McCook-Summit storage basin are estimated using the following assumptions:

1. 57,000 acre-feet of storage is provided below a level 100 feet below the ground surface.[3/] This corresponds to an in-place rock volume of 92,000,000 cubic yards.

2. The surface area of the basin at ground level is approximately 350 acres and the overburden thickness is approximately 25 feet, thus the quantity of overburden to be managed is an estimated 14,000,000 cubic yards. Of this, 5,000,000 cubic yards is sludge and the remainder is soil material.

3. The material between the overburden and the 100-foot storage level is mined rock, and its in-place volume is estimated as 34,000,000 cubic yards.

4. The difference between the final volume, after compaction, at the mountain site, and the volume before excavation is defined as the swell factor. Swell factor for mined rock is assumed to be 20 percent and that for overburden is assumed to be zero.

5. The in-place density of the dolomite bedrock is estimated as 2.16 tons/cu.yd.; that of the residual soil is 1.62 tons/cu.yd.; and that of the sludge is 0.9 tons/cu.yd.

6. The sludge is initially about 22 percent solids by weight. The final volume, after drying following disposal, is not estimable unless the method of incorporating it into the soil is specified. However, the actual final volume is relatively inconsequential insofar as the cost of handling sludge is concerned.

The quantities of materials requiring management from other areas in the region are calculated to be equal in volume to the tunnel, or pipeline, or storage facility from which they originate. The swell factors for mined rock and moled rock are assumed to be 20% and that for residual soil is assumed to be zero. The quantities of materials can be summarized as follows:

Mined rock from suburban sites is either zero or 31,000,000 tons, depending on the alternative.

Moled rock from the urban area is approximately 30,000,000 tons.

Moled rock from the suburban and rural areas varies from zero to 70,000,000 tons, depending on the alternative.

Residual soil from the suburban and rural areas varies from 6,000,000 to 82,000,000 tons, depending on the alternative.

Logistics

The important logistical problems associated with rock management include crushing, loading, transportation, unloading, and placement of the material. Crushing and loading are common to many of the potential management options for mined rock. The drilling and blasting process creates a non-uniform distribution of rock sizes; this rock must be crushed in order to facilitate loading. The loading facility design will depend upon the construction schedule. Local quarries have facilities which can load up to 1000 tons/hr., but some coal loading facilities in this country are capable of up to 3,000 tons/hr.

Transport to the mountain site can be either by truck, rail or barge. Truck transport is not practical between the McCook-Summit site and the mountain site due to the amount of material and the comparatively long distances. Rail transport has been considered for similar use with comparable quantities, and appears to be the most economical at the present time. A one-way trip from the McCook-Summit area to the mountain site is estimated to be 20 miles by rail. Barge transport is attractive if the mountain site is adjacent to a waterway, but this is assumed not to be the case. Another possible transport method is by pipeline. It is not competitive at the present time, partly due to the requirement for three stages of crushing, followed by grinding, to make the rock acceptable. Placing the materials at the mountain site could be accomplished by either trucks or conveyors, or combinations of the two.

Moled rock does not require crushing prior to loading. Since the quantities are smaller, less sophisticated loading facilities are needed. Transportation from the source of rock to either the local disposal site or a rail loading facility is assumed to be by truck. rban rock destined for the mountain is then transported by rail to the site. All movement of materials in the rural and suburban areas is assumed to be by truck, since the quantities are relatively small and the distances short.

Component Design

The design of the mountain landscape must provide for the optimum conditions for recreational use. It must serve as a focus for outdoor activities for millions of people and thus have areas specially designed for a variety of uses. Some assumptions are made of the gross characteristics of its design for the purpose of this study. It is assumed to be roughly the shape of a pyramid with 6:1 side slopes. All of the overburden from beneath the mountain is removed in order to bear the weight of the mountain directly on the bedrock. This soil, and the residual soil from the McCook-Summit site, are used as covering for the rock core. The estimated height of 430 feet is below the height specified by the Federal Aviation Administration as being a danger to aircraft. 2/ Removing or flattening the top would not add greatly to the circumference if a lower mountain were to be desired. The land area covered by the mountain is approximately 600 acres.

The smaller rural and suburban landscapes are assumed to be 100 feet in height and to have side slopes of 8:1. Each would cover about 59 acres of land. They could have a variety of uses, depending upon the goals of the local communities. The urban landscapes would probably have such a variety of configurations that no typical size or shape was assumed. For those rock sources near the irrigation areas for land treatment wastewater management alternatives, the rock is used in the construction of the lagoon dikes and in the facing of irrigation drainage ditches.

RECREATIONAL ISLANDS IN LAKE MICHIGAN

The recreational demands on the Lake Michigan shoreline in the C-SELM region are such that consideration has been given to the construction of an extensive group of islands in Lake Michigan along the shore. 4/ 5/ Besides adding attractive and useful open space to the area, proper

design could provide a large area of protected water for boating and could reduce dramatically the erosion of shoreline. Construction of the islands is, of course, contingent upon satisfactory proof that their construction will not adversely effect the lake ecology, currents or other factors. The discussion here regards the feasibility of using the rock and soil from construction of the wastewater management system for the construction of such islands in the event that the decision is made to construct them.

Estimated Quantities

The basis of design of the islands provides that the mined rock and residual soil from construction of the McCook-Summit storage basin and two-thirds of the moled rock from tunnel construction in the urban area are used for the islands. This is the same basis as was used for the mountain landscape. The remainder of the materials are used for a variety of urban landscapes and rural and suburban hills less than 100 feet in height, as is discussed in the mountain landscape section.

Logistics

The most feasible existing method of moving materials from the urban area to the sites of islands is by barge. The crushing and loading requirements for barge hauling are similar to those discussed previously for rail transport. Special traffic patterns may be necessary to allow use of the existing waterway, depending upon the construction schedule which is implemented.

Another possible mode of transport from the McCook-Summit site to the islands is through the use of an underground rail or conveyor system through the main conveyance tunnel. If this tunnel is constructed prior to the excavation of the storage basin, it could be used in this manner. The design calls for a tunnel up the Sanitary & Ship Canal to Lake Michigan at the mouth of the Chicago River. Extension of the tunnel to the island site could provide a transport path for the materials. Upon completion of construction, this same tunnel could be used for transport of pedestrians by rail or people-mover to the islands.

Moled rock from within the urban area is transported from its source to barge loading points along the waterway, from which it is transported by barge to the island sites. All movement of materials in the rural and suburban areas is again assumed to be by truck, as it is in the mountain landscape option.

Component Design

No detailed designs are available for recreational islands in Lake Michigan. The only reference with regard to island construction in the lake is a report on an airport in the lake. 6/ The airport was to be constructed using polders made of sand and rock. The rock was assumed to be quarried from an area on the lake's bottom near the airport.

The circumstances are too different for the island construction and the airport construction to get an island cost from the airport study. Some assumptions are made here about the islands in order to get a rough estimate of the costs that may be involved. It is assumed that islands, not polders, are constructed. Thus, assuming a water depth of 30 feet, approximately 2,000 acres of islands can be constructed from the rock and overburden originating in the urban area. Construction of polders would enable substantially more usable area to be constructed.

It is also assumed that the islands are one-eight mile in width and one-half mile in length. The first stage in the construction process is the encirclement of the island area with sheet piling to allow unwashed materials to be dumped inside thereby minimizing possible pollution of the lake. Thus, transportation and sheetpiling are the only items included in island construction costs due to the uncertainty of proper island design considerations.

MOUNTAIN LANDSCAPE WITH COMMERCIAL USE OF ONE-HALF OF THE ROCK FROM THE MC COOK-SUMMIT SITE

This option is identical to the mountain landscape option, except it is assumed that one-half of the rock from the McCook-Summit storage basin is stockpiled and eventually sold at the prevailing market price. The remains of the rock and overburden from the urban area would be managed as described in the mountain landscape section. The amount of rock to be sold is considered to be a realistic quantity that can be marketed or stockpiled during the construction period.

Estimated Quantities

The quantities are identical to those for the mountain landscape option, except one-half of the rock from the McCook-Summit site is removed for the mountain and one-half is removed for stockpiling and eventual sale.

Logistics

It is assumed that the rock for sale is transported to an existing quarry in the McCook-Summit area where it is graded and/or stockpiled for use as the market demand allows. All other logistics are identical with those in the mountain landscape option.

Component Design

The component design is identical with that in the mountain landscape option, except that the size of the mountain is reduced to approximately 350 feet.

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4.   Johnson, Johnson & Roy, Inc., A Study of the Future of Chicago's Lakefront, 1972.

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G.   REUSE SYSTEMS

INTRODUCTION

This section on reuse systems presents, in detail, the basis of design of various system components for both the potable reuse provision and the recreational - navigational reuse provision.   In addition, a section on the impact of the reuse system on key parts of the water resource is presented.

Potable Water

A discussion on projected potable water deficiencies and a tabulation of the deficiencies per-township over the entire C-SELM area is presented for the design years 1990 and 2020.   Two methods of supplying the deficiencies are presented as Options 1 and 2.   Option 1 discusses use of different sources of water to supply the potable need.   Option 2 proposes to supply these needs with water from Lake Michigan only.

Withdrawals of Lake Michigan water are limited and are under the control of the International Joint Commission of the United States and Canada.   Present ruling of the U.S. Supreme Court limits the withdrawal rate for the Chicago region to 3,200 cfs.   With proper evidence as to dire need of fresh water for this region, and the evidence of best management of present supplies, this ruling may be appealed and Option 2 of potable water supply would become viable.

Recreational and Navigational Flows

Streams within the C-SELM area require minimum base flows to permit usage for recreational purposes.   In the same context, the maximum desirable flow in a given stream should also be investigated. To meet minimum flow requirements, flows are supplied to area streams. The points at which flows are added for recreational purposes are called injection points.   These points are selected on the basis of best service for recreational needs within the C-SELM area.   Actual flows supplied to injection points are between minimum and maximum desirable and are based on total flow available for reuse.   Recreational flows are supplied from reclaimed M & I flows from either treatment plant or land treatment systems.

It should be noted that maximum desirable flows are calculated with respect to present dimensions of stream channels, and are less than bank-full rates of these channels. Flood flows are controlled by the stormwater control systems proposed throughout the C-SELM region.

Navigational requirements are based on lockage needs. They are provided by a closed pumping system which successfully prevents Lake Michigan waters from interacting with river system waters.

Reuse Impacts Considerations

Reuse of renovated wastewater in the C-SELM area for the purposes mentioned above has great influence on a number of interrelated factors. Quality of flows, recreational possibilities and potable water supplies are among these factors.

POTABLE REUSE

Introduction

The necessity of an adequate potable water supply for satisfying all the needs of a community is self-evident. Potable water should be free from turbidity, color, odor and from any objectionable taste, and should be of a reasonable temperature. There is also a more exacting set of quality specifications that quantitatively define potable water quality in terms of chemical and biological constituents. Such water is termed potable or "drinkable" meaning that it may be consumed in any desired amount without concern for adverse effect on health. Long-range plans for any community must by necessity include planning for the supply of this high-quality potable water resource at a reasonable price.

Long-range plans of communities within the C-SELM study area recognize this need and outline possible future sources of supply. The sources are: Lake Michigan water, groundwater, surface water, and renovated M & I flows. However, the Lake Michigan source has a Supreme Court limit on its availability; groundwater sources are being exhausted by continuous pumpage; surface water is polluted; and the M & I water, treated to meet existing water quality standards, cannot be used for potable needs.

The balance between the demand for water and the available supply is already upset in some parts of C-SELM region. This imbalance will become more pronounced as the C-SELM design target dates of 1990 and 2020 are approached. The magnitudes of these balances in potable water supply versus water needs are discussed below.

The Illinois State Water Survey has conducted studies on this subject and has published supply availability projections or yields.[1][2] These results are used in conjunction with the C-SELM study of water

demands for municipal, commercial, industrial, and agricultural con-
sumption. The projected potable supply deficiencies are established
from a combination of the data and are presented in Table B-IV-G-1
for the various C-SELM townships for the years 1990 and 2020.

Thus the potable reuse system is designed to satisfy these de-
ficiencies from whatever sources available within the C-SELM area,
based on which option is under consideration. A number of sources
exist.

Present Water Supply Sources And Service Areas

Presently, there are two basic water supply sources in the
C-SELM area; (a) Lake Michigan water and (b) groundwater obtained
from wells. The two areas which are served by the above sources are
identified as; a) present Lake Michigan service area, and b) present
groundwater service area. Figure B-IV-G-1 identifies both of these
service areas.

Future Water Supply Sources

In order to supply the increasing demand for potable water, all
the available water supply sources will have to be fully developed and
appropriately utilized in the future.

These sources are: (a) Lake Michigan water, (b) groundwater
obtained from wells, (c) surface water renovated via the rural storm-
water management systems, and (d) renovated municipal and industrial
flows from treatment plants or land treatment sites.

Future Potable Water Distribution Options

The C-SELM study contemplates two options by which potable
water deficiencies can be satisfied.

Option 1. Recognizing the legal limitation on the diversion of
not more than 3200 cfs of Lake Michigan water for all uses within the
Illinois portion of C-SELM, Option 1 places great emphasis on the
utilization of all other available sources of potable water. However,
before these sources and their impacts are explained, a discussion on
the actual amounts of Lake Michigan water available for diversion is
in order.

Due to the reversal of the Chicago River flow from Lake Michi-
gan to the Sanitary and Ship Canal and other man-made channels, much
of the stormwater runoff, which historically flowed into Lake Michigan,

Table B-IV-G-1

POTABLE WATER DEFICIENCIES BY TOWNSHIP
TARGET YEARS 1990 - 2020

No.[a]	Township Name	Deficiency 1990 MGD	Deficiency 2020 MGD	No.	Township Name	Deficiency 1990 MGD	Deficiency 2020 MGD
32	Antioch	0	0.4	62	Lyons	1.4	9.7
33	Lake Villa	0	0.6	82	Wheatland	0	3.7
34	Newport	0	0	83	DuPage	0	8.4
35	Benton-Zion	5.3	12.6	64	Lemont	11.9	14.9
37	Avon	0	3.3	65	Palos	1.2	5.5
38	Warren	1.0	5.8	84	Plainfield	0	3.3
41	Fremont	0	0	85	Lockport	12.3	15.9
42	Libertyville	5.4	16.6	86	Homer	0	8.7
45	Ela	0	0	68	Orland	6.1	12.2
46	Vernon	2.1	5.6	69	Bremen	12.1	22.0
49	Palatine	10.9	17.6	70	Thornton	0.9	5.1
50	Wheeling	17.1	26.9	66	Worth	1.1	6.6
51	Northfield	6.4	14.6	87	Troy	0	5.0
54	Shaumburg	13.0	21.3	88	Joliet	16.3	31.5
55	Elk Grove	20.8	27.6	89	New Lenox	0	5.9
73	Wayne	0	7.3	90	Frankfort	0	2.8
74	Bloomingdale	7.5	18.2	71	Rich	10.8	16.5
75	Addison	18.0	29.8	72	Bloom	35.9	44.9
76	Winfield	2.9	14.7	91	Channahon	0.5	0
77	Milton	15.9	28.1	92	Jackson	0	0
78	York	14.8	20.1	93	Manhattan	0	0
79	Naperville	0.3	9.8	94	Green Garden	0	0
80	Lisle	8.6	19.2	95	Monee	0.6	8.2
81	Downers Grove	11.1	17.7	96	Crete	0	1.3
					TOTAL DEFICIENCIES	272.2	549.9

a Reference to key map of Figure B-IV-G-1

LIMITS OF C-SELM AREA

NUMBERS	COUNTY
1-16	McHENRY
17-31	KANE
32-47	LAKE
48-72	COOK
73-81	DU PAGE
82-104	WILL

AREA ENCLOSED BY METROPOLITAN SANITARY DISTRICT

AREA SUPPLIED BY LAKE MICHIGAN WATER

SCALE OF MILES
0   5   10   15
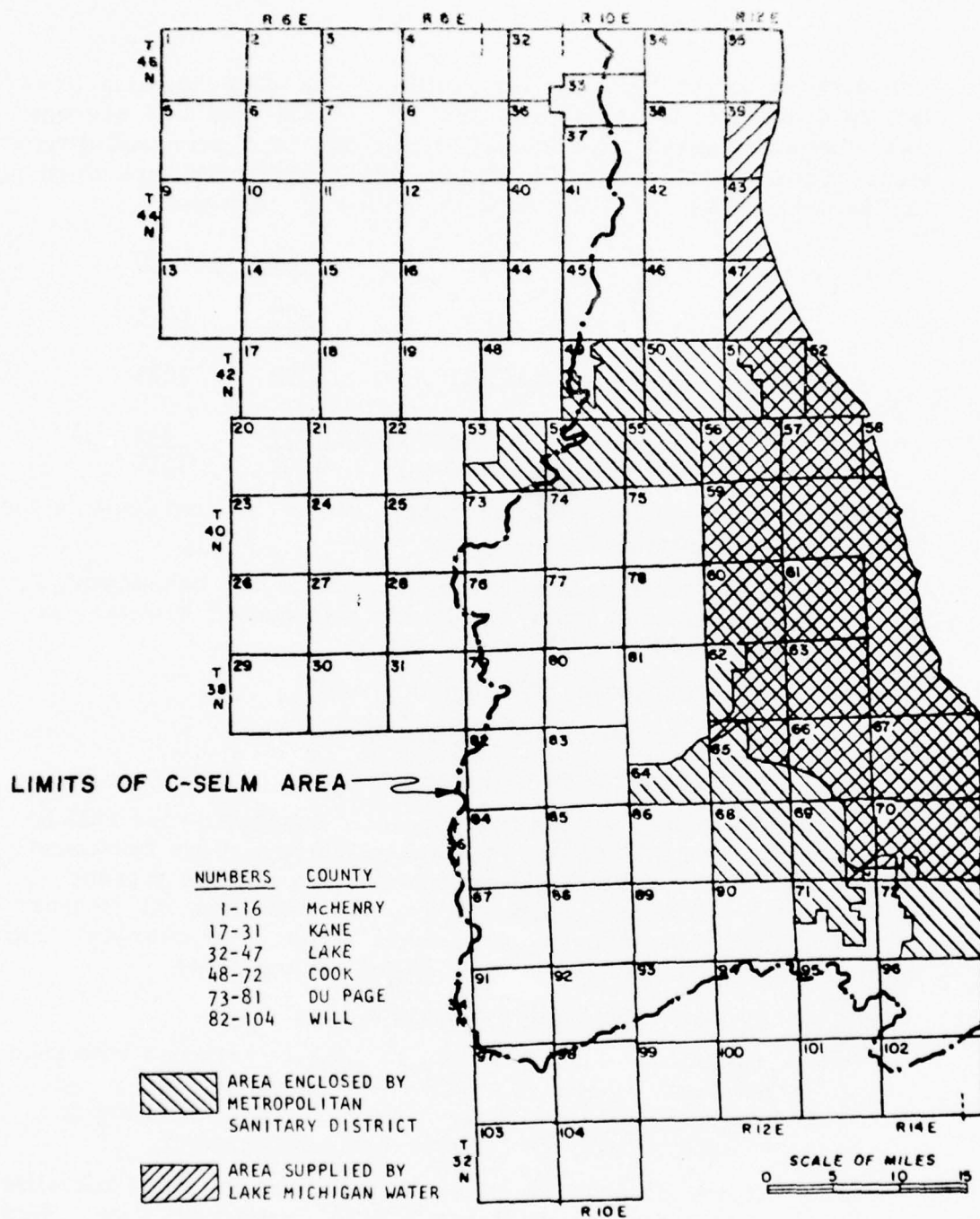
NOTE: USE WITH TABLE B-IV-G-1

Figure B-IV-G-1
KEY MAP
POTABLE WATER DEFICIENCIES BY TOWNSHIP

B-IV-G-5

was diverted to the Illinois river system. The total diversion flows legally allotted to the Illinois portion of C-SELM take into account *this stormwater runoff and subtract it from the total permitted diver-sion*. Consequently, actual available withdrawals from Lake Michigan for the years 1990 and 2020 show the following figures:

	Flows in MGD	
	1990	2020
Limit on Total diversion (3200cfs)	2068	2068
Indirect (stormwater)diversion	477	488
Available as water supply	1591	1580

Only the flows available as water supply can be counted upon in the Option 1 considerations.

There are other sources of supply, which can be integrated into an over-all potable water supply and management system. As previously mentioned, these sources are:

a. Goundwater obtained from wells

b. Renovated rural stormwater

c. Renovated M & I flows

Management of all the potable water supply sources will be explained on the basis of two independent service areas mentioned before, the present Lake Michigan service area and the present groundwater service area. The supplies available from all sources are distributed between these two service areas. For example, Lake *Michigan* water is supplied to both of the service areas.

Present Lake Michigan service area

Potable need for this area is mainly satisfied with Lake Michigan water. However, as demand increases and the available source becomes exhausted, the need for alternate sources of supply becomes apparent.

In the case of the Lake Michigan service area, the alter-nate source of water is the M & I renovated flow. This flow can be obtained either from plants in the Treatment Plant mode, or from land sites via return tunnels in the Land Treatment mode of water renovation. Both sources will be discussed.

Additional supplies from Plant M & I flows. Total
demands for potable water for the target years as
well as available flows for the present Lake
Michigan service area are tabulated below:

	Flows in MGD	
	1990	2020
Total Supply		
Lake Michigan water	1492	1411
M & I flows	100	321
Total Demand	1592	1732

This means that a total of 100 MGD in 1990 and
321 MGD in 2020 of reclaimed M & I flows would be
mixed with Lake Michigan water to meet need require-
ments. The City of Chicago Central filtration plant would
draw 61.4 to 197 MGD from the Chicago River while
the City of Chicago South filtration plant would draw
38.6 to 124 MGD via pumpage from an MSD treatment
plant.

Additional supplies from land treatment sites. In
the land treatment mode, M & I flows would be returned
to the C-SELM area via a system of return tunnels.
However, the amount of flow in these tunnels would vary
because of the seasonal nature of the land treatment
system. All average yearly flows are returned in a
period of eight months or in ratio of 1.5:1. Conse-
quently the mixing of Lake Michigan and M & I flows
is also on a seasonal basis. This means that during
the 8-month period when maximum flows are returned
from land sites, Lake Michigan water withdrawal is
reduced in proportion. In the remaining 4-month
period, the return systems would carry no potable reuse
flows; consequently, Lake Michigan water withdrawals
would compensate for the reduced M & I flows. The
following tabulation illustrates this concept:

	1990		2020	
	summer	winter	summer	winter
Total Supply				
Lake Michigan water	1442	1592	1250	1732
M & I land site flows	150	–	482	–
Total Demand	1592	1592	1732	1732

Mixing of flows will occur at the two filtration plants as discussed in the previous paragraph. It must be brought out here that the restriction on 3200 cfs (2068 MGD) diversion is on an average yearly basis. It can be exceeded for any one day, or whatever length of time is necessary for diversion of greater amounts of flow. The yearly average however, must be maintained.

Present groundwater service area

Potable needs for this area are satisfied from all four sources previously mentioned. Several points must be mentioned now for clarification.

1. Not all the rural stormwater available in 1990 is utilized for potable water needs. This is done in order to retain the same distribution system for 1990 and 2020. Because the system is designed for 2020 flows, all 2020 reclaimed rural stormwater is utilized.

2. While some need points are supplied with rural stormwater alone, others draw from reclaimed rural stormwater and M & I flows from the adjacent streams.

3. All rural stormwater from the management sites is released to nearby streams, unless a local need warrants an independent conveyance line from the rural site to the point of need.

4. The distribution system for the year 2020 is adopted for year 1990 even though smaller flows will initially be pumped.

5. Needs of all points which receive rural stormwater, alone or in combination, are satisfied by pumping stormwater during the eight summer months and withdrawing water from ground or other sources at accelerated rates for the four winter months.

6. All pressure lines and pumping stations supplying rural stormwater directly to need areas are designed for 1.5 times the average yearly flows from the stormwater management sites. This is based on an eight-month summer flow and a four-month winter flow.

7. There are two pumping systems which supply Lake Michigan water to concentrated, groundwater service-area need points. In the 2020 flow regime, these points are not satisfied with rural stormwater alone and other sources of supply had to be found

8. The pumping systems just mentioned will operate in two different modes depending on treatment technology used. For plant systems, mixing of M & I flows is on an average yearly basis. In land systems, the 1.5 factor for an 8-month pumping period is operational.

With the above points in mind, a tabulation of water demands in the plant mode of treatment was prepared and is presented below.

Lake Michigan water and M & I flows are utilized within the two pumping systems mentioned in item 7 of the clarifying statements. Groundwater and rural stormwater satisfy the needs of all other deficiency points, and vary with seasons by a factor of 1.5, as mentioned previously.

	Flow in MGD			
	1990		2020	
	summer	winter	summer	winter
Total Supply				
Lake Michigan water	99	99	169	169
Groundwater from wells	140	348	94	496
Rural Stormwater	208	–	402	–
M & I Reuse	33	33	113	113
Total Demand	480	480	778	778

B-IV-G-9

In similar fashion, a tabulation of water demands in the land treatment mode is presented below:

	Flow in MGD			
	1990		2020	
	summer	winter	summer	winter
Total Supply				
Lake Michigan water	82	132	112	282
Groundwater from wells	140	348	94	496
Rural Stormwater	208	-	402	-
M & I Reuse	50	-	170	-
Total Demand	480	480	778	778

Figure B-IV-G-2 shows the water distribution piping of the Option 1 potable reuse system, including the two Lake Michigan water pumping systems. The location of deficiency points within the township boundaries is discussed in the following Option 2 description. Where the deficiency point is too far away from the nearest stream, a pressure line and pumping station is provided. In cases where a deficiency point happens to be near a stream, only a pumping station has been considered.

Option 2. In Option 2 there are no restrictions placed on the amount of diversion from Lake Michigan. Consequently all potable water deficiencies are satisfied solely with Lake Michigan water. Rural stormwater flows and all M & I renovated flows enter C-SELM waterways and are used only for recreation and navigation purposes. For the purpose of this study, the physical location of a water supply deficiency point for each township is established at the geographic center of the township. However, if all of the township is not urbanized by the year 2020, then the deficiency point is located at the center of the projected area of development. These points then serve as receivers of the potable water distribution system. Responsibility for the delivery of water from the receiver points to individual points of need within the township is, for the scope of this study, given to the local governing units. Delivery of water from Lake Michigan to the individual receiver points is within the scope of this report and

WISCONSIN
ILLINOIS

Harvard

MC HENRY CO.

Marengo

Fox R.

LAKE CO.

WAUKEGAN

East Fork

Middle Fork

West Fork

Des Plaines R.

EVANSTON

ELGIN

KANE CO

N. Branch

Chicago R.

CENTRAL
FILTRATION
PLANT

COOK CO.

CHICAGO

Salt Creek

DU PAGE CO.

SOUTH
FILTRATION
PLANT

Lake Michi

AURORA

Fox R.

KENDALL CO.

DuPage R.

Ship Canal

Calumet Sag Channel

Chicago Sanitary

EAST CHICAGO

GARY

Little Calumet R.

JOLIET

Jackson Creek

Crown Point

Illinois R.

WILL CO.

Morris

ILLINOIS
INDIANA

LAKE CO.

GRUNDY CO.

LEGEND

———————— LAKE MICHIGAN WATER LINES

— — — — RURAL STORMWATER SUPPLY LINES

□ MUNICIPAL AND INDUSTRIAL TREATED WATER PUMPING STATION

—•—•— MUNICIPAL AND INDUSTRIAL TREATED WATER PRESSURE LINE TO FILTRATION PLANT

△ PUMPING STATION DIRECTLY TO NEED POINT

● NEED POINT

Figure B-IV-G-2

OPTION I - POTABLE REUSE SUPPLY SYSTEM

B-IV-G-11

is addressed below. Deficiencies of potable water for the target years 1990 and 2020 were discussed earlier and are listed in the following tabulations:

Year	Deficiency in MGD
1990	272.2
2020	549.9

With an ultimate target date of 2020, it is proposed that all pressure lines and pumping stations be designed to have capacity to handle the 2020 flows. Economies can be gained, however, if the following is taken into consideration:

a. Trunk lines that handle 2020 flows only, can be built at a later date and not with the original system which must meet 1990 requirements as well.

b. Although the pumping station proper must be designed for 2020 flow characteristics, an economy may be realized by installing pumping capacity for 1990 level and increasing it later to 2020. Power consumption will, of course, reflect both levels of pumpage.

Figure B-IV-G-3 shows the physical layout of five pumping systems which deliver Lake Michigan water to 41 receiver points scattered throughout the Illinois portion of C-SELM.

Each pumping system serves a number of receiver points. As few as one or as many as 13 points are served by an individual pumping system. Pressure lines are laid out to follow the public rights-of-way to avoid the need of obtaining access right from private individuals. Booster pumping stations are added where head loss economics dictate. The Indiana portion of the C-SELM study area is considered to have adequate groundwater and Lake Michigan water supplies to satisfy all present and future needs through the year 2020.

Conclusions

Potable water deficiencies for the target years of 1990 and 2020 are computed and methods of satisfying these needs are discussed. Four sources of water of satisfactory quality exist within the C-SELM area to fulfill all the requirements for potable water in both target periods. Two options of delivering the water to points of deficiency are discussed and shown in respective exhibits.
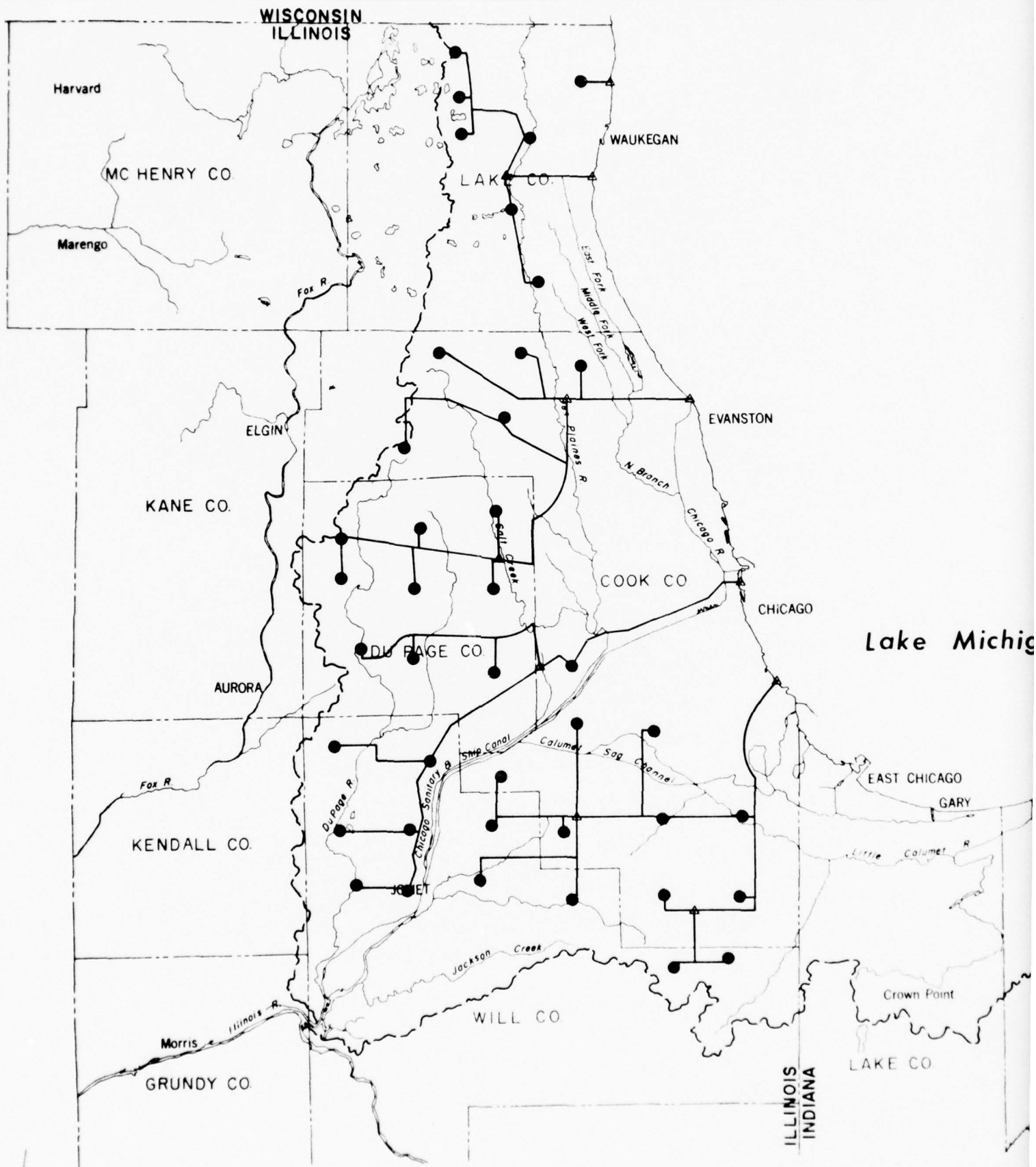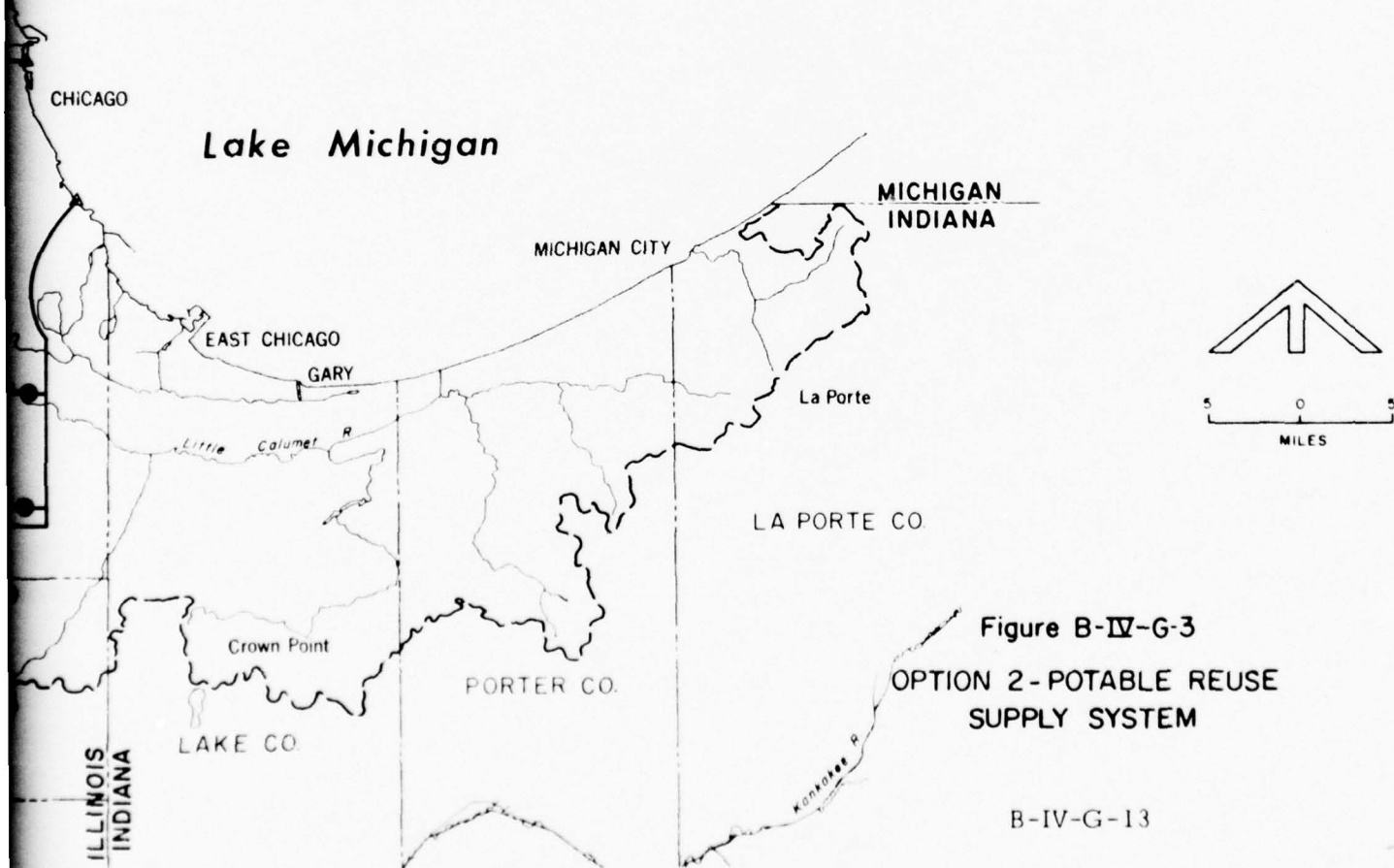
Figure B-IV-G-3

OPTION 2-POTABLE REUSE
SUPPLY SYSTEM

RECREATIONAL - NAVIGATIONAL REUSE

General

The purpose of this portion of reuse description is to establish the basis of design for the recreational and navigational provisions of the overall reuse system. Two key determining factors are: 1) the recreational-navigational flow basis, and 2) the location of recreational-navigational flow injection points. The recreational-navigational reuse flow basis remains the same for all alternatives and reuse options which incorporate this provision. However, the logistics of supply change between alternatives.

The following discussion presents the basis of design for recreational flows and navigational flows. This is followed by a description of the planned flow injection scheme. Following that is a discussion of the basis of design for component parts of the respective reuse systems.

Recreational-Navigational Flow Determination

Recreational Recreational flows were established through a reiterative process which worked between minimum and maximum allowable flow boundaries. Flows were selected for injection at specific points and compared against the minimum and maximum flow at that point. Figure B-IV-G-4 presents a map of the C-SELM area which shows the recreational-navigational reference points. Minimum and maximum flows were established by different criteria.

Design criteria for minimum recreational flows were established as the 60 percent median flow, which translates to a flow that is exceeded on a yearly average 60 percent of the time. Table B-IV-G-2 lists minimum flows at 40 of the total 45 key recreational injection points.

Maximum allowable flows in the C-SELM area streams were established on a basis of permissible flow which would not cause bank erosion. The flow was computed by using the results of a statistical analysis on the correlation between the flow, velocity, and drainage area performed on the Des Plaines River. The methodology of this analysis is presented in Data Annex B, Section IV-G. River bed materials in the study area are generally composed of glacial drift. The permissible velocity for streams flowing through this type of soil is quite varied. The estimated range for this velocity is 1.5

EVANSTON

CHICAGO

Lake Michigan

MICHIGAN
INDIANA

MICHIGAN CITY

EAST CHICAGO
GARY

La Porte

Little Calumet R

LA PORTE CO.

5   0   5

MILES

Crown Point

Figure B-IV-G-4

PORTER CO.
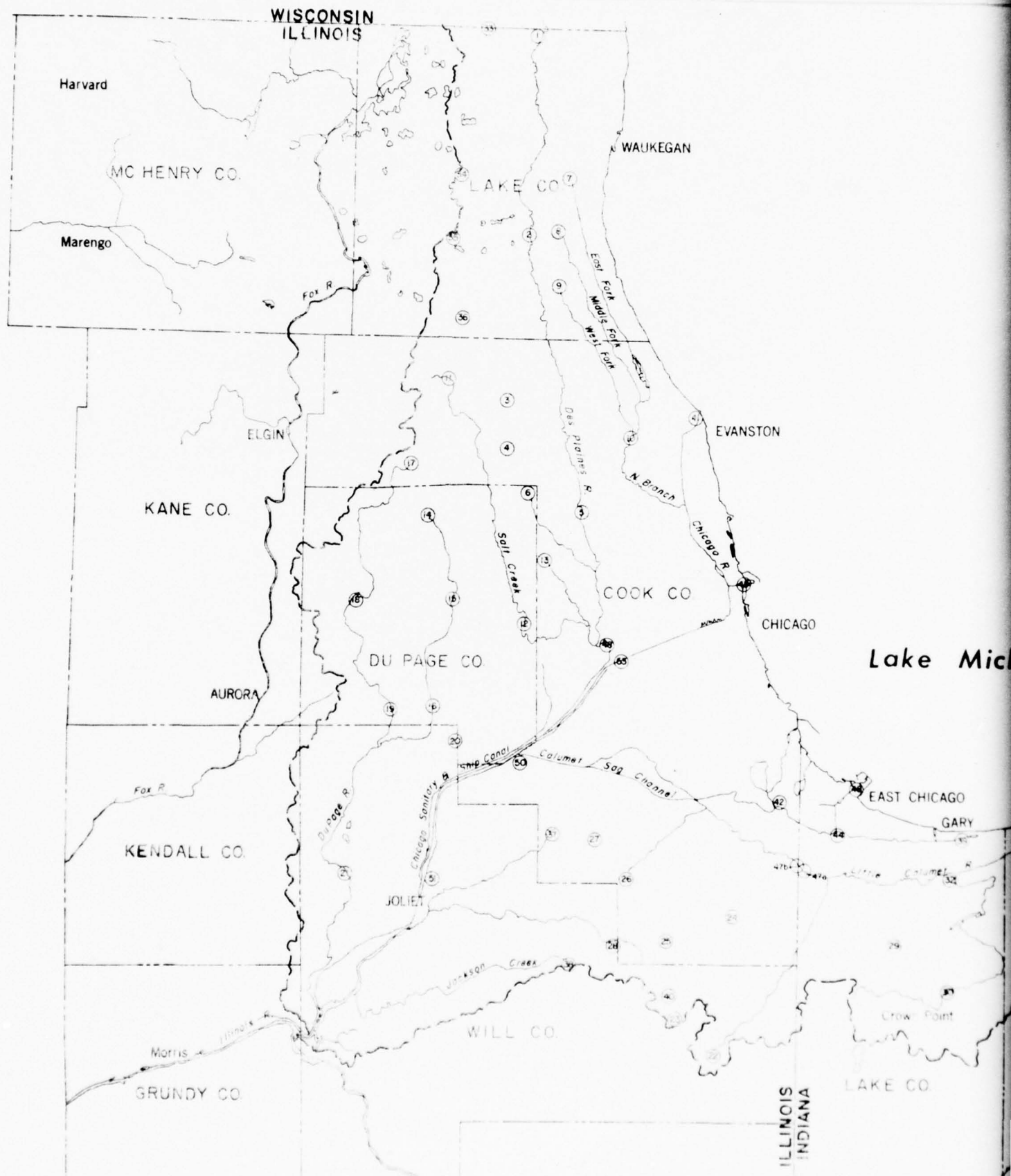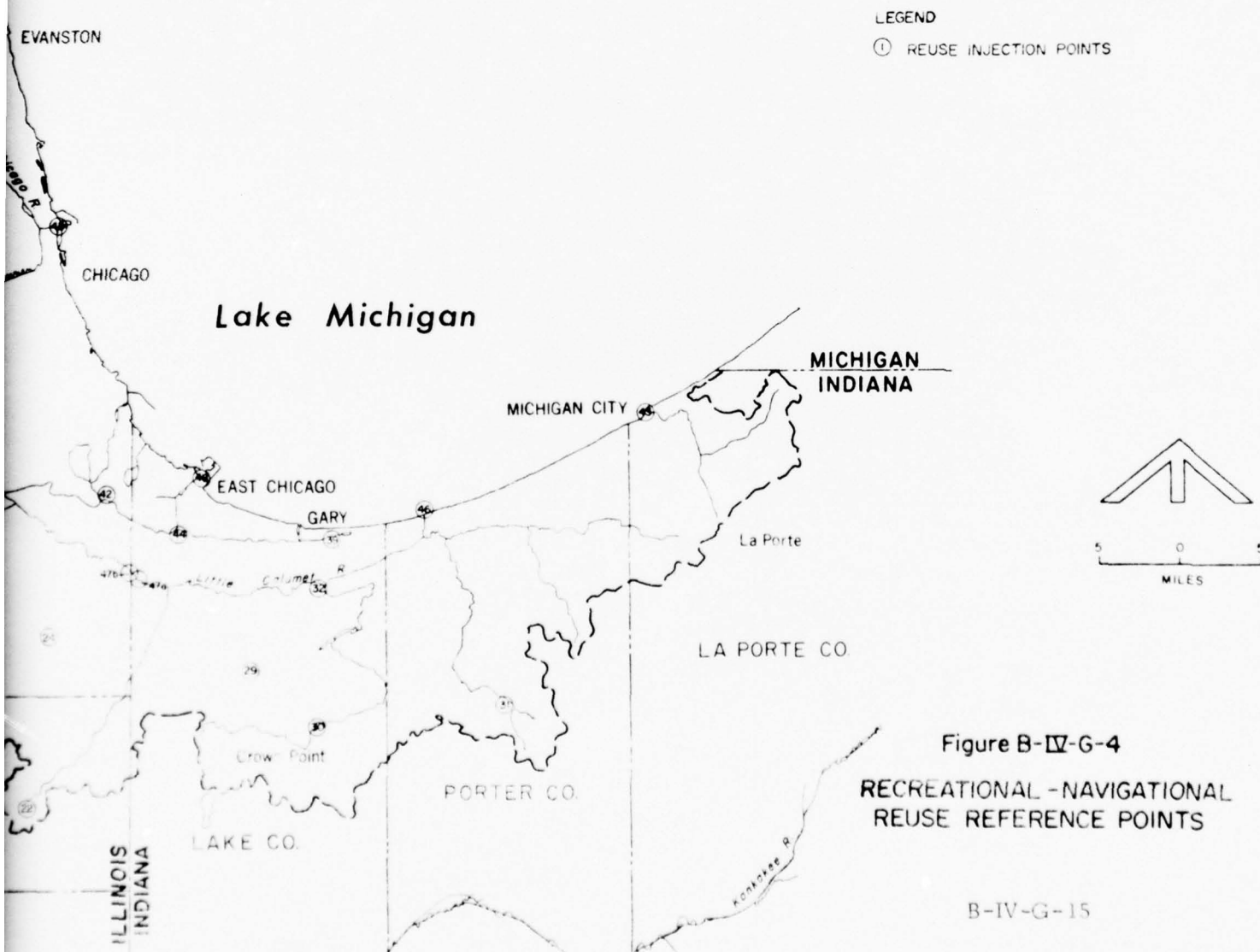
RECREATIONAL -NAVIGATIONAL
REUSE REFERENCE POINTS

ILLINOIS
INDIANA

LAKE CO.

B-IV-G-15

Table B-IV-G-2
RECREATION & NAVIGATION FLOWS

Map Reference No.	Location	Reuse flow MGD		
		Minimum	Supplied	Maximum
1	Upper DesPlaines River [a]	4	35.6	235
2	Middle DesPlaines River [a]	6	--	340
3	Weller Creek [a]	2	5.7	38
4	Willow Creek [a]	2	3.6	24
5	Lower DesPlaines River [a,c]	20	--	500
6	Silver Creek [a]	2	3.4	23
7	East Fork, N.Branch, Chicago R. [a]	3	5.0	15
8	Middle Fork, N.Branch, Chgo R. [a]	2	11.8	23
9	West Fork, N.Branch, Chgo R. [a]	2	11.8	23
10	North Branch, Chicago River [a] from pt 5	3	100.0	210
11	Upper Salt Creek [a]	2	10.4	69
12	Lower Salt Creek [a]	12	--	223
13	Upper Addison Creek [a]	2	6.3	42
14	Upper E. Branch, DuPage River [a]	3	6.3	25
15	Middle E. Branch, DuPage River [a]	3	20.1	84
16	Lower E. Branch, DuPage River [a]	2	--	165
17	Upper W. Branch, DuPage River [a]	2	7.7	51
18	Middle W. Branch, DuPage River [a]	4	--	109
19	Lower W. Branch, DuPage River [a]	2	34.5	230
20	Upper Lily Cache Creek [a]	4	7.4	49
21	Lower Lily Cache Creek [c]	22	-25.0	410
22	Upper Hart Ditch, Little Calumet R. [a]	2	5.1	34
23	Upper Deer Creek, Little Calumet R. [a]	2	6.0	40

RECREATION & NAVIGATION FLOWS

Map Reference No.	Location	Reuse flow MGD		
		Minimum	Supplied	Maximum
24	Lower Thorn Creek, Little Calumet R. [a]	14	--	90
25	Upper Butterfield Creek, Little Calumet River [a]	2	13.1	87
26	Upper Calumet Slough, Little Calumet River [a,b]	2	9.0	60
27	Upper Tinley Creek, Little Calumet R. [a,b]	2	4.1	27
28	Hickory Creek [a]	9	11.3	112
29	Upper Turkey Creek, Little Calumet R. [a]	2	10.5	70
30	Mid-Upper Deer Creek, Little Calumet River [a]	3	18.0	120
31	Upper Salt Creek [a]	2	5.1	34
32	Lower Salt Creek [a]	34	40.0	400
33	Upper North Mill Creek [a]	2	8.7	58
34	Avon-Fremont Drainage Ditch [a]	2	5.0	33
35	Indian Creek [a]	2	3.0	20
36	Buffalo Creek [a]	2	4.8	32
37	Spring Creek [a,b]	4	5.0	17
38	Jackson Creek [a]	3	6.3	42
39	Upper Grout Calumet River [a,b]	4	5.0	20
40	Upper Thorn Creek [a]	2	6.0	40
41	Chicago River Evanston [a]	-	20.0	--
42	Calumet [a]	-	75.0	--
43	Michigan City [d]	-	--	--
44	Grand Calumet, Indiana Harbor [a]	-	10.0	--

RECREATION & NAVIGATION FLOWS

Map Reference No.	Location	Reuse flow MGD		
		Minimum	Supplied	Maximum
45	Chicago River Locks [b]	-	35.6	--
46	Outlet of Burns Harbor [d]	-	--	--
47 a,b.	Stateline Little Calumet [a]	-	20+20	--
48	Indiana Harbor Outlet [d]	-	--	--
49	DesPlaines River, Upstream of Salt Creek [c]	-	--	650
50	Calumet Sag, at San & Ship Canal Control Point [a]	-	--	--
51	Lockport Locks [c], from pt. 21	-	25.0	--
52	Chicago Sanitary & Ship Canal d.s. from W.S.W. Treatment Plant [c]	-	100.0	--
53	DesPlaines R. Upstream of confluence with Kankakee R. [d]			

[a] Injection Point

[b] Recirculation Point

[c] Transfer Point

[d] Control Point

to 4.0 feet per second. For positive protection against stream erosion in the study area, a permissible velocity of less than 2.0 feet per second should be used. A value of 1.8 feet per second was selected for this study. Maximum allowable stream flow values were then established for the 40 recreational injection points. These values are also shown in Table B-IV-G-2.

Actual recreational flow determinations are also listed in Table B-IV-G-2. As indicated earlier, the quantity of flow was selected to fall between the minimum and maximum flows established above. A minimum dispersion treatment plant alternative (17 service areas) was selected as the base alternative for the determination of all reuse flows. In addition, flows were established for the 1990 flow basis. A detailed analysis of stream flows was made in conjunction with the determination of actual recreational flows. The purpose of the additional analysis was to establish if the accumulated flow from injection point reuse flows and treatment plant overflows would overtax the maximum stream capacity.

This was accomplished by a cumulative summary, or routing of flows through the C-SELM stream system. There are four locations where flow actually leaves the study area. These have been defined as control points, and are referenced on both Table B-IV-G-2 and Figure B-IV-G-4 as points 43, 46, 48, and 53. Flow was continuously summed to each of these control points, and the quantity of the flow was checked against the allowable stream capacity.

In order to check maximum flow in the stream-routing analysis, maximum conditions were selected. The maximum condition was established as a flow occurring during the summer months immediately following a storm period in the 2020 design year. These maximum flows include reclaimed municipal and industrial flows as well as reclaimed urban, suburban, and rural stormwater flows. This maximum flow condition reflects the potable water Option 2, the no-diversion limitation situation. This introduced the rural stormwater flows into the stream system with no withdrawal for potable use. In addition to these flows, recreational injections were also included. The recreational flows were initially selected by trying to maximize the use of the 1990 dry weather flows available within any particular service area and supplying this flow to a nearby injection point. In isolated situations where dry-weather flows were not economically available, flows were recirculated within the same watershed. Points where this has been done are identified as recirculation points in addition to injection points.

When cumulative flow quantities exceeded the maximum permissible stream flow value, as established above, flows were transferred out of the particular basin to an adjacent water course which could accept additional flow. These points have been identified in Table B-IV-G-2 as transfer points. After transfers, the routing was continued.

This cumulative flow routing was carried out with the initial assumption on the values for recreational injections. After the complete routing, flow values were restudied on an overall basis to analyze the balance of flows in all streams. If the flows were greater or less than the existing flow regimes, the recreational injections were changed to establish new flow quantities. A second complete routing was then made to observe the effect of the changed recreational injection on the overall flow regime. In addition to flow routing, an analysis of stream depths in several streams was performed to test depths resulting from the minimum and maximum conditions discussed above. The results of this analysis are tabulated and enclosed for reference in the Data Annex B, Section IV-G, Table BA-IV-G-1. This pattern was repeated until a flow pattern was established which met the criteria of sound engineering judgment.

Minimum flow conditions were then checked against the final recreational flow conditions to see if these were met in every case. Minimum flows for routing reflected the 1990 dry-weather, winter flow conditions, which is the minimum flow seen in the alternative management system.

As pointed out earlier, the general methodology of flow routing was initially established from a base alternative which reflected flows from 17 service areas. Specific routings and flow summaries for each of the wastewater management alternatives are presented in Appendix D.

Navigational. Navigational flows are based upon individual lockage requirements. This is a reflection of the actual number of lockage in any specific period. For this basis of flow selection, it is assumed that all Lake Michigan lock operations are placed on a closed, pump-back system. In this design, therefore, lockage flows are not required. The flow in the streams from recreational injections, municipal and industrial flows, and all reclaimed stormwater flows, provides adequate depths for navigational shipping.

Recreational-Navigational Design Requirements

Recreational. Recreational flows are distributed to the various injection points, transfer points and recirculation points through a system of force mains and pumping stations. The system varies between type of alternative, i.e., treatment plant system or land treatment system.

Treatment plants.

Flows from treatment plant alternatives are pumped from the plant location to an injection point. This is done on the basis of the established final recreational flow as shown in Table B-IV-G-2. All other flow from a particular plant is allowed to overflow at the plant location. All lines for movement of recreational reuse flows are sized for an assumed velocity of six feet per second. The size of the line and the friction head loss are then selected from the Pipe Friction Manual of the Hydraulic Institute. With a known length of the pipe, total head loss is computed. Pumping station selection is made on the basis of the total head loss and other assumed friction losses.

Land treatment.

For the land treatment alternatives, flows were first returned from the land treatment sites by return conveyance tunnels. The return flows varied between summer and winter as discussed in the Drainage discussion presented in Data Annex B, Section IV-A. Flows were brought back to access points and distributed to the injection points through a force main pumping system exactly the same as the one used for the treatment plant alternatives. Since 12 months of flow is returned in an 8-month period (less stored winter reuse flows), and only limited flows are needed to satisfy the recreational injection needs, excess flows during the 8-month period are overflowed to the stream system.

Return tunnels are sized on the basis of the maximum period of flow return from the land treatment system. This value is equal fo the maximum drainage system capacity of six inches per week from each land treatment site, as outlined in Appendix B, Section IV-A.

Tunnel sizing and pump station design criteria are
determined upon the same basis as addressed in
Appendix B, Section IV-E.

Navigational. In addition to the pump-back, closed-lockage
system mentioned above, an air-bubbler system, designed to prevent
any mixing between reuse flows in the streams and Lake Michigan at
the lock interface, is provided. This system is provided at each end,
or gate area, of the locks. A diffusion system is envisioned. Com-
pressors deliver air to a bubbler manifold located on the bottom of the
lock, at a rate sufficient to establish a barrier to intermixing.

Pump-back facilities are designed to empty or fill the standard
lock chamber within a period of four to five minutes.

REUSE IMPACT CONSIDERATIONS

Water Balances

Water balances serve the purpose of graphically displaying the
movement of key flows within and between the Illinois and Indiana
portions of the C-SELM study area. This is particularly important
with respect to Illinois and the 3200 cfs restriction placed upon the
state's Lake Michigan withdrawal. In addition, the water balances
serve to identify the actual quantities of these flows when applied to
a specific wastewater management alternative as is done in Appendix
D.

Water balances are also important in establishing the basis of
design with respect to flows in the reuse section described earlier.

Figure B-IV-G-5 is a graphical presentation of the many factors
important in the water balance study. Three key nodes appear on this
water balance for each state. These are, (1) present Lake Michigan
service area, (2) present groundwater service area, and (3) Illinois or
Indiana treatment facilities.

A system of flow indicators trace the interaction of these key
nodal points. These flow indicators show all of the key elements of
water movement as follows:
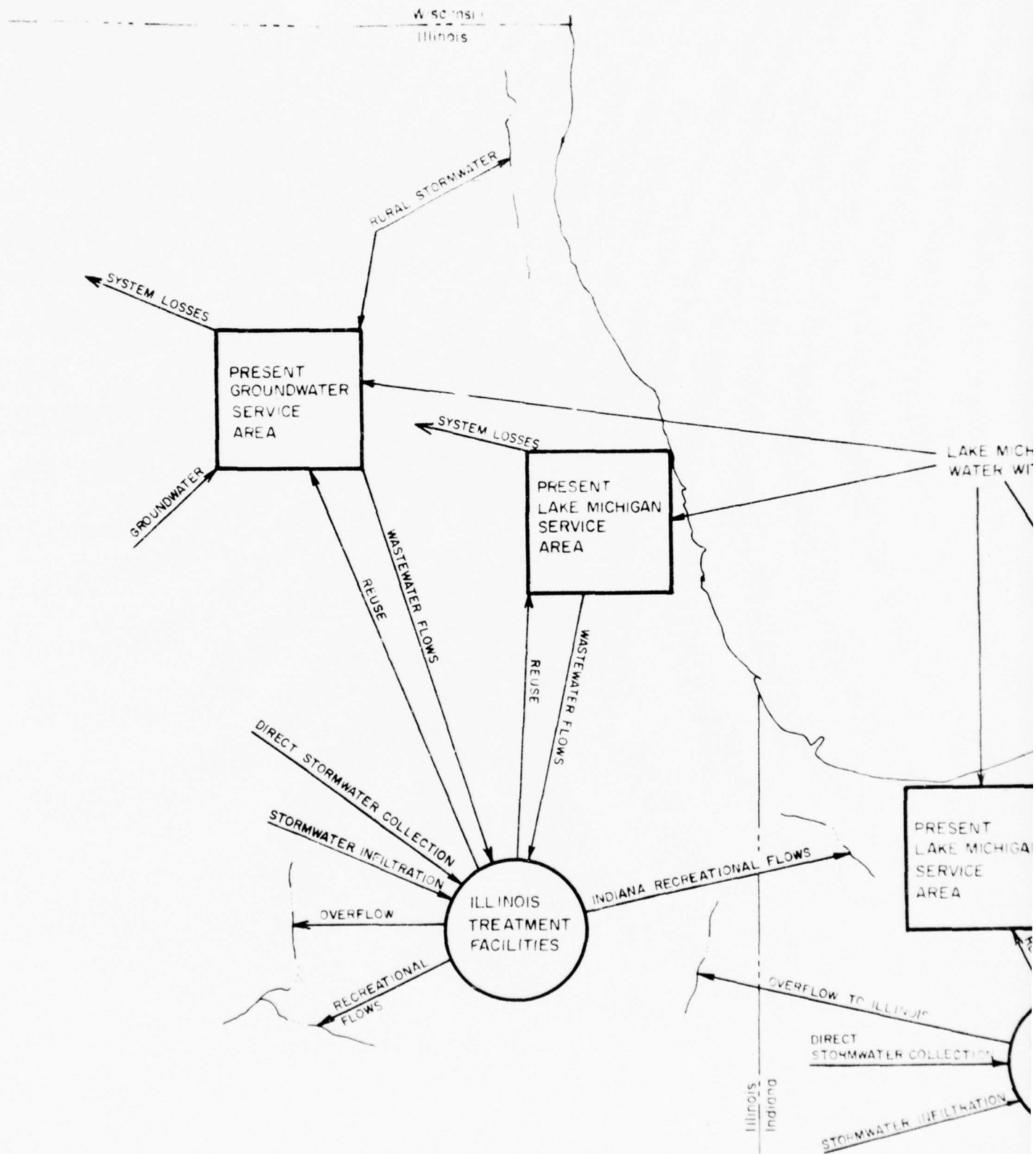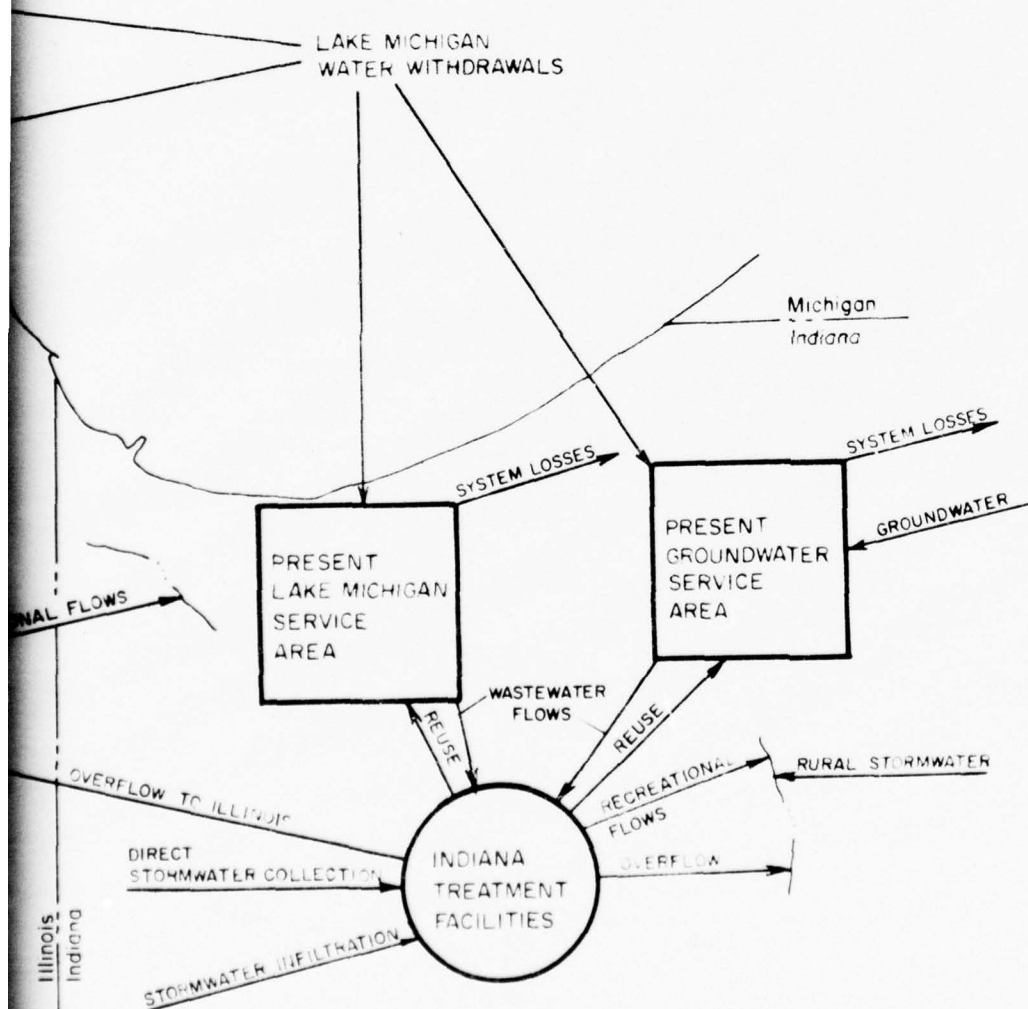
Figure B-IV-G-5
STYLIZED WATER BALANCE

1. Municipal and industrial supplies and supply sources which include:

    a) Lake Michigan

    b) Groundwater

    c) Rural Stormwater

    d) Reuse

2. Municipal and industrial supply system losses.

3. Untreated wastewater flows.

4. Direct collection of urban and suburban stormwater

5. Infiltration of urban and suburban stormwater

6. Reuse flows, including

    a) recreational

    b) municipal and industrial

7. Treatment system overflows at outfalls

8. Rural stormwater flows

The water balance diagram in Figure B-IV-G-5 is detailed for each of the four AWT alternatives in Appendix D. Separate balances are presented for 1990, with and without a 3200 cfs restriction on Lake Michigan withdrawal, for each of the alternatives.

In addition to the graphical water balance, a companion table is presented which reflects the eight key items outlined above.

Water Quality

The water quality considerations in reuse can be separated into two parts, one for potable reuse and another for recreational reuse.

Potable. A water quality concern with respect to the reuse of reclaimed flows for potable supplies is the concentration of total dissolved solids (TDS). TDS also impacts on the possibility of return of reclaimed flows to Lake Michigan. This question will be addressed later in this section.

The values for TDS concentration found in each source of potable supply is different. In order to balance the TDS concentration

a mix of flow supplies was established within the present Lake Michigan service area and the present groundwater service area. The purpose was to balance the two service areas with respect to TDS concentrations.

The present TDS concentration in Lake Michigan is approximately 160 mg/l. Regional AWT reclaimed water TSD concentrations are in a range between 500 and 535 mg/l, depending on the treatment technology in question. Rural stormwater and groundwater concentrations are projected to be 130 mg/l. Actual flow mixes and the resulting overall TDS concentrations are outlined for each alternative for each of the two key time periods, 1990 and 2020, in Appendix D.

Recreational. Recreational water quality concerns center mainly around the establishment of a viable aquatic community, and the creation of an aesthetic visual resource. The aquatic considerations are greatly enhanced through the establishment of permanent flows of high quality reclaimed water in the area streams. The prevention of stormwater flows from reaching the stream system helps to insure the quality of the streams established through the injection of recreational reuse flows.

In addition to established flow regimes, it is equally important to consider the quality of the water which is being reused to establish the aquatic community. Any action which might interrupt or over stimulate the natural food chain in the aquatic community at any level is serious to all organisms higher up on the chain. The nutrient or "fertilizer" concentration in the various NDCP reclaimed waters associated with the treatment technologies presented in this report are not so low or so high as to cause this type reaction in the stream system. Another concern in any aquatic system is the suffocation of aquatic organisms by lowered or completely removed oxygen concentrations. For this reason the BOD and oxygen relationships of the reclaimed water are very important. This is of no concern in the systems envisioned for reuse since, in addition to the extremely high quality of the effluent with respect to these particular parameters, the very action of delivering the waters to the streams enhances their dissolved oxygen content. For example, additional aeration could be accomplished by an injection point mechanism which induces further aeration by passing water over a series of steps, or small rapids as it leaves the injection pipe.

If a viable aquatic community is established, the possibility
for a natural aesthetic resource is greatly enhanced. The return of
game fish and other water creatures will encourage the maintenance
of stream banks in a clean condition and the return of the attention
of area residents to these valuable resources.

Return of Flows to Lake Michigan

It is important to consider the return of reclaimed flows to
Lake Michigan. This is especially true in light of the 3200 cubic
feet per second diversion limitation for Illinois withdrawal. Without
the return of flow to Lake Michigan, the reuse of rural stormwater
and municipal and industrial flows in potable reuse option becomes a
necessity for the Illinois C-SELM area - in the not too distant
future.

Current formal EPA policy disallows the return of reclaimed
flows to Lake Michigan because of the associated TDS concentrations.
Current Lake Michigan values for TDS are in the range of 160 mg/l
as reported by the City of Chicago. The Federal EPA holds that the
reintroduction of Illinois C-SELM reclaimed water at a concentration
of 500 mg/l will violate the non-degradation policy of the pertinent
federal water quality legislation.

Recent analyses of the TDS concentrations at the water intakes
of the City of Chicago indicate an annual increase of between 1.13
and 1.38 mg/l.[4] Information from a lake-wide survey of Lake Michi-
gan was compared to the City of Chicago information.[4][5] This data
was limited to a single time period (1962-1963), and consisted of over
400 lake-wide tests. The data falls somewhat below the City of
Chicago information. Calculations based upon total inflows from the
16,500-sq. mi. land drainage area tributary to the 10,500 sq. miles
of the lake from about Milwaukee and Muskegon south to the end of
the lake indicate the present average annual rate of increase to be
about 1.0 mg/l. The following assumptions were used in the calcula-
tion:

Inflow or Outflow	Average rate, MGY	T.D.S. mg/l
Runoff from land to lake (mostly Michigan watersheds 10" from 16,500 sq. mi.)	2,950,000	300
Rainfall on lake (30" per year on 10,500 sq. mi.)	5,600,000	25
Evaporation from lake (30" per year on 10,500 sq. mi.)	5,600,000	0
Outflows from the lake (assumed to equal the inflow from land)	2,950,000	160

(The lake depth is assumed to be an average of 300 feet. Therefore, the volume is 10,500 sq. mi. x 640 acres x 300'/3 = 670,000,000 MG. The annual flow volumes into and out of the lake are seen to be fractions of one percent of the lake volume.) If the 1990 flows from C-SELM of 3,000 MGD at an assumed TDS of 500 mg/l were to be added to the lake, the additional incremental change in TDS would be about 1.09/670 x (500 - 160) = 0.55 mg/l. Added to the present average of about 1.0, the increase would be 1.6 mg/l per year.

Chemical standards for drinking water quality, as established by the 1961 revision of the Public Health Service Drinking Water Standards, lists a recommended maximim limit for TDS at 500 mg/l. The World Health Organization sets the potable limit for TDS at 750 mg/l. Many areas in the western parts of the United States have 1,000 to 3,000 mg/l of TDS in their potable supplies. A threshold concentration (a value which might normally not be deleterious to fish and other aquatic life) of 2,000 mg/l TDS has been established as an upper bound for healthy aquatic life in fresh water.[3]

It is evident that a great deal of difference exists between the present TDS in Lake Michigan and the higher levels of TDS considered acceptable by many organizations. Nevertheless, if the total quantity of dissolved solids discharged into Lake Michigan per year remains as at present, and the management of flows into and out of the lake also remains unchanged, a steady rise in the TDS concentration in the water in the lake can be projected such that a concentration of 500 mg/l could be reached in several hundred years.

There are ways in which such an inevitable rise in T.D.S. could be mitigated. For example, Lakes Michigan and Huron are now at all-time highs such that substantial lowerings would be desirable. If all discharges into the lower 40% of Lake Michigan could be avoided during such wet periods, some reduction in maximum lake level could be achieved and the average rate of increase in TDS could be substantially reduced.

During periods of low lake level it would be desirable to return treated flows to the lake to mitigate the effects of low lake level. Return of say, 3,000 MGD to the lake for 10 years adds a volume equivalent to one foot of depth over the approximately 50,000 square miles of Lakes Michigan and Huron. This foot could be useful and beneficial during such dry periods. Assuming a pattern of 10 years at above-average lake levels followed by 10 years of below-average lake levels, the program envisaged here could produce the following differences in rates of TDS increase:

	Present	Proposed	TDS
Average inflow from land, MGY	3,000,000	1,500,000	300
Average inflow C-SELM treated wastewater, MGY	0	500,000	500
Average inflow, precipitation, MGY	5,600,000	5,600,000	25
Average outflow, MGY	3,000,000	2,000,000	160
Average evaporation, MGY	5,600,000	5,600,000	0

The corresponding increments per year of TDS would be:

	Present	Proposed
Inflow from land @300 mg/l	+1.34	+0.67
C-SELM treated wastewater @500 mg/l	0	+0.37
Precipitation @25 mg/l	+0.21	+0.21
Outflow @160 mg/l	-0.72	-0.48
Evaporation @0 mg/l	0	0
	+0.83 mg/l	+0.77 mg/l

The proposed program is seen to reduce the rate of TDS concentration increase per year from 0.83 to 0.77 mg/l. The proposed program would also reduce the fluctuation of the levels of Lakes Michigan and Huron by an appreciable amount, reducing the financial losses associated with both high and low lake levels.

The proposed program presented here is by no means a complete program for managing the levels of Lakes Michigan and Huron. The development of such a complete program is beyond the scope of this study. But the idea presented here demonstrates that management of lake levels can be associated with lowering the average rate of increase of TDS in Lake Michigan, while at the same time permitting the return to Lake Michigan of treated C-SELM flows when the lake levels would be low.

The return of reclaimed flows to Lake Michigan is an extremely important consideration and should be seriously investigated in future policy and decision-making.

All other Metropolitan areas located in proximity to the great lakes, including Milwaukee, Wisconsin and Gary, Indiana, return their secondary treatment effluents to Lake Michigan.

The NDCP-quality reclaimed flow contemplated by this study would be a marked improvement in quality over existing secondary effluent return flows. For example the algal growth potential, or eutrophication strength of the treatment plant and land treatment NDCP-quality return flows would be one-fourth and one-fortieth respectively, of that existing in the 80% phosphorus removed secondary effluents currently piped to Lake Michigan from the Indiana C-SELM area. The land treatment return flow quality is estimated to be comparable to Lake Michigan natural background water quality with respect to eutrophic strength, meaning that land treatment return flows would not increase Lake Michigan algal activity beyond historic, natural levels.

While small amounts of TDS accumulation could be credited to Illinois C-SELM return flows to Lake Michigan over a long time period, TDS is not an algae-producing nutrient; furthermore, it is not possible, based upon any current understanding of aquatic ecology, to project any problem associated with this degree of TDS accumulation.

In summary the TDS question is particularly troublesome because, as a result, the C-SELM design must undertake to keep all of Illinois' reclaimed water out of Lake Michigan, while, at the same time, the United States Supreme Court limits the Illinois diversion from Lake Michigan. This also influences the C-SELM design to return all Indiana reclaimed water to Lake Michigan. Not only does this appear to be an inconsistent policy, but it adds materially to the cost of reuse systems envisaged in this study.

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IV. COMPONENT BASIS OF DESIGN

H. SYNERGISM SYSTEMS

The synergism systems, outlined in Appendix B, Section II, have been discussed in very broad terms. Among these, however, pumped storage and dissipation of waste heat from power generating stations warrant more detailed design descriptions. These systems are described, in detail, in the following section.

PUMPED STORAGE AND WASTE HEAT DISSIPATION

Energy Forecast

Predictions of future energy requirements have been given in a number of studies, and in most cases these energy forecasts have been based on the assumption that the large increases in energy needed in the next 50 years would be substantially provided by the generation of electric power.

One such forecast is the Federal Power Commission's (FPC) National Power Study [1] compiled in 1966-68. This study forecasts that the minimum energy demands would require a doubling of installed capacity every ten years for the next 30 years. This prediction was based on a projection of the energy demands experienced over the past 20 years, including an allowance for decreasing population growth rates. Using this projection the minimum commitment for generating facilities in the year 2000 would be approximately seven times the 1973 levels.

In another study, Professor Earl Cook [2], professor of geography and geology at Texas A & M University, pointed out the possible need to conserve resources to minimize the associated pollution problems and to maintain adequate reserves for future generations. This reasoning lead to a leveling-off of power consumption at about the year 2010 and 2020 at a level of about four times the present 1973 level. To achieve the result envisaged by Professor Cook would require unprecedented public policy changes, as it would call for an arbitrary limitation of consumption of resources.

The C-SELM estimate for this study, presented in Figure B-IV-H-1, assumes it would be wise to make a minimum commitment for generating facilities at about seven times 1970 levels for the year 2020, or 65,000 MW as compared to 12 times present levels as predicted by local power companies. This estimate falls between the two projections listed above, although it does seem to fit more nearly with the conservative projection of Professor Cook.

Design Basis

The utilization of land treatment storage lagoons for the dissipation of the waste heat generated during the production of electric energy has been investigated for the C-SELM study. In making this investigation, certain basic assumptions and design criteria have been established for the practical application of this type of system.

The first and primary assumption made is that nuclear power generating stations located near the land treatment sites would provide the additional power needed to meet the energy requirements of the C-SELM study area through the year 2020. An additional 55,000 MW of electric generating capacity would be required to supplement the existing or 1970 installed capacity of 10,000 MW in order to meet the 65,000 MW projected requirement in 2020 according to the energy forecast of the previous section. The waste heat to be dissipated from the generation of this much power amounts to 8,780 billion BTU's per day. The cooling pond surface area required for dissipating this heat is approximately 70,000 acres, provided that the temperature of the cooling pond is allowed to exceed 80°F during the summer. This surface area requirement for heat dissipation is on the same order as the surface area provided by the land treatment storage lagoons in the year 2020. Thus, the storage lagoons provided by the land treatment alternative could be fully utilized as part of the waste heat dissipation system for the generation of power at the projected 2020 levels.

Figure B-IV-H-2 shows the general arrangement of a modular land treatment system in combination with a nuclear power generating system. The wastewater being pumped into the treatment system through the inflow shaft will be distributed to the aerated lagoons for biological treatment. The effluent from the aerated lagoons will be directed to the sedimentation lagoons where most of the suspended solids will be removed. The effluent will then be directed into the storage lagoon and the sludge will be stored in the sludge lagoon for thickening and storage. The thickened sludge will be applied to the land allocated for sludge application. The power station cooling system

| POPULATION (IN MILLIONS) | 7.33 | 8.29 | 9.24 | 10.10 | 11.70 | 12.17 |
| MW / CAPITA | 1.39 | 3.00 | 4.60 | 6.20 | 7.80 | 9.50 |
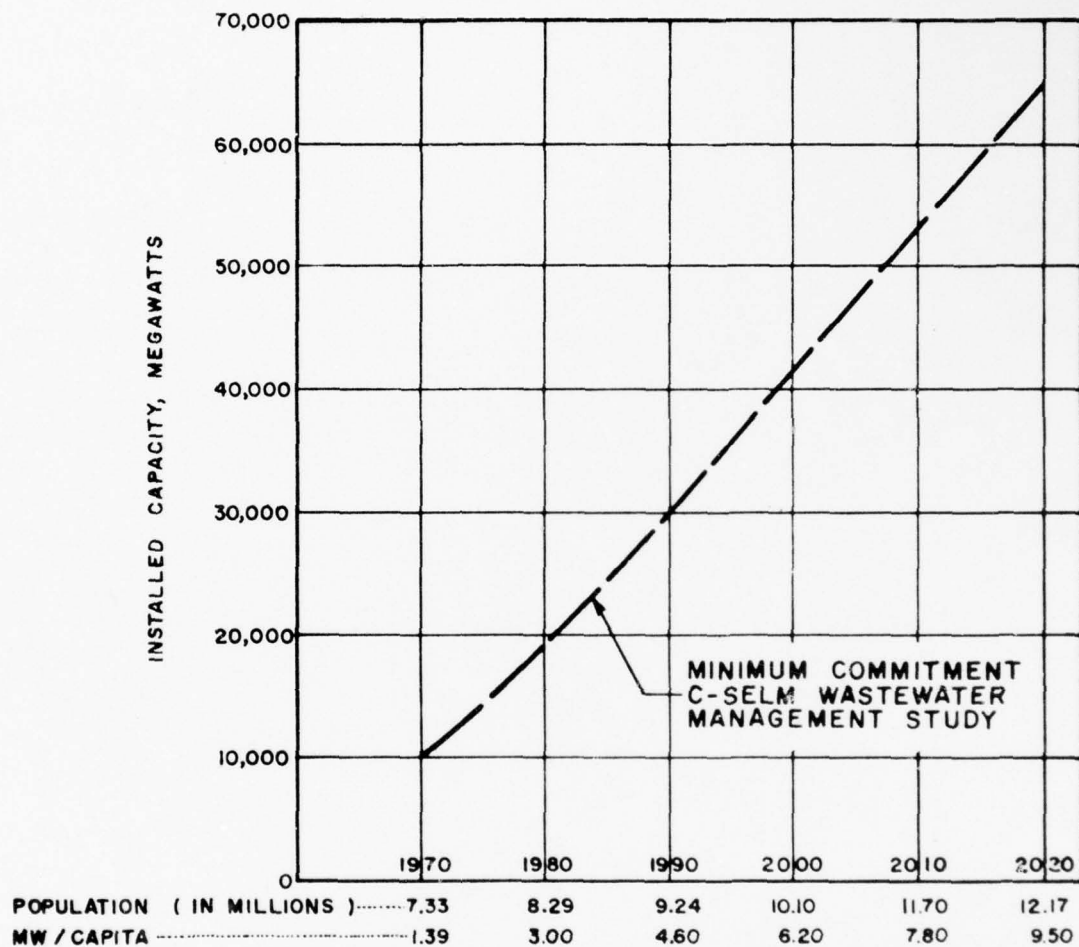
Figure B-Ⅳ-H-I

ENERGY FORECAST FOR C-SELM AREA

B-IV-H-3

REACTOR
GENERAT
AERATION
SEDIMENTATION
WASTEW
AERATION LAGOONS
SLUD
SLUDGE LAGOONS
STOR

REACTOR
GENERATOR

WASTEWATER INFLOW SHAFT

SLUDGE APPLICATION

FLOW

INTAKE STRUCTURE

PUMPED STORAGE
DROP SHAFT

STORAGE LAGOON
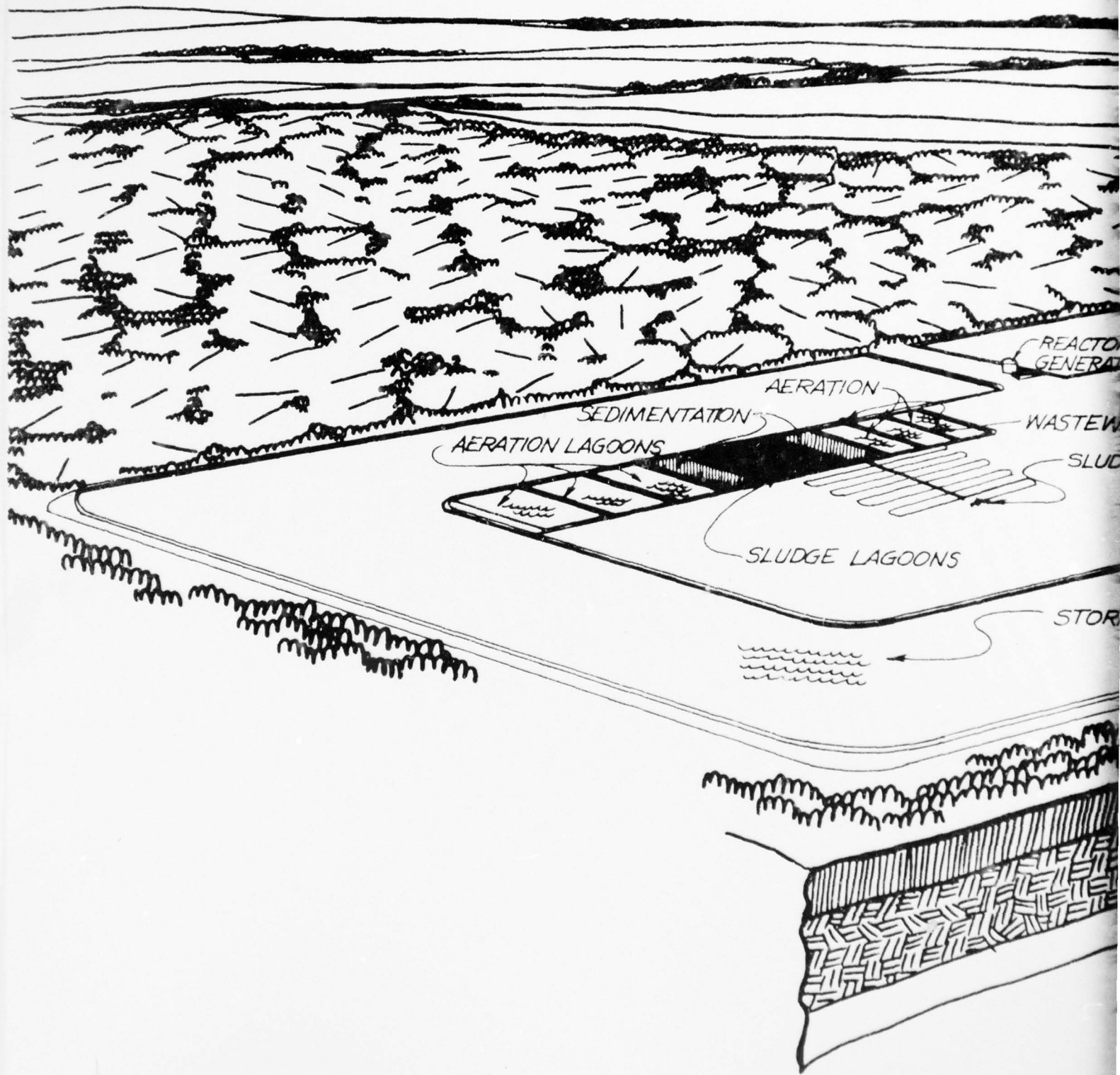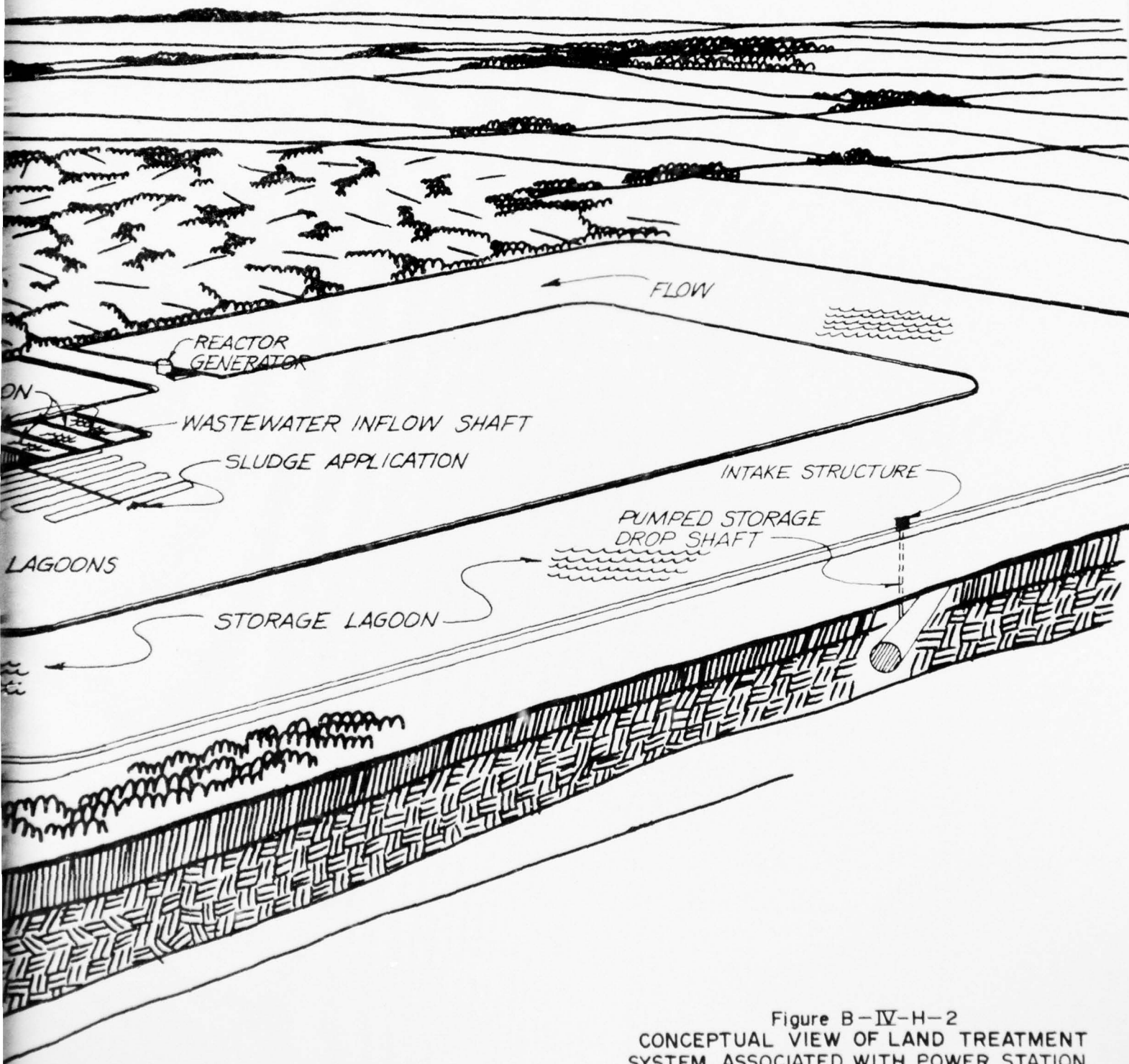
LAGOONS

Figure B–IV–H–2
CONCEPTUAL VIEW OF LAND TREATMENT
SYSTEM ASSOCIATED WITH POWER STATION

would pump cool wastewater from one end of the lagoon and discharge heated wastewater into the other end. The total make-up water required to replace the water lost by evaporation during this heat dissipation process is estimated to be 43 MGD or approximately 16% of the average daily flow coming into the land treatment system.

In analyzing the utilization of a pumped storage power generating system in conjunction with the land treatment alternative of the C-SELM study, the following basic design criteria have been determined. The total dead storage provided in the land treatment storage lagoons is estimated to be 15,000 Ac - ft for each module . This volume of stored wastewater could be a large upper reservoir in a pumped storage system. Assuming that the lower reservoir would be constructed 1,000 feet below the ground surface in the suitable Galena dolomite formation and that the duration of power generation would be eight hours, a pumped storage power station could have a maximum generating capacity of 1.4 million kilowatts. This capacity is calculated using an overall efficiency of 70%. The peak flow in the penstocks would be 22,500 cfs, and the equivalent penstock area should be a minimum of 1150 square feet to avoid the undesirable conditions which could be produced at velocities in excess of 20 fps. The incremental 55,000 MW of new thermal electric generating capacity would probably be associated with a need for an incremental ten million kilowatts of pumped storage peaking capacity. Thus, perhaps only half of this potential synergistic benefit would be developed.

BIBLIOGRAPHY

1.   Federal Power Commission, <u>The Methodology of Load Forecasting</u>, 1969.

2.   Earl Cook, "Scientists Plumb the Unknowable Future", <u>C & EN</u>, Jan. 10, 1972.

IV. COMPONENT BASIS OF DESIGN

I. NON-STRUCTURAL MANAGEMENT

CONTROL OF SOIL EROSION

Soil erosion is important in wastewater management because of the effect of sedimentation in water bodies, in streets and ditches, and in culverts and sewers. Sediment becomes a pollutant when it fills lakes, reservoirs, streams, ditches and culverts, and degrades water quality with plant nutrients, insecticides, herbicides, heavy metals, and possibly bacteria and viruses. Sediment is costly to process in water and wastewater treatment and to remove from lakes, reservoirs and streams. It also contributes to a loss of recreational and aesthetic values of water and land resources, and increases flooding.

Sediment produced by erosion is the most extensive pollutant of surface water in the United States, estimated at from 500 to 700 times the loading from sewage effluent. It is especially pronounced in the West and Southwest and some parts of the East. It is not nearly as critical in the C-SELM area.[1]

Erosion and sedimentation is accelerated when the soil mantle is disturbed and vegetation and surface litter is removed. Agricultural development is said to increase erosion 4-to-9 times, and construction activity to increase the sediment yield 10 times over cultivated land, 200 times greater than land in pasture, and 2,000 times greater than land in timber.[2] The degree of erosion depends on the type of soil, the slope of the terrain, the amount of rain and wind, the time of exposure, and the construction and conservation practices employed. Control of erosion and sedimentation is a matter of taking these factors into account and planning with them in mind.

Special measures are necessary to control erosion of stream banks, which may be threatened when streams carry increased amounts and rates of runoff in urbanizing areas and when channel straightening or realignment exposes new materials. The stability of the bed and banks must be evaluated under the predicted flow conditions and it may be necessary to dissipate the energy of stream flow by decreasing slopes and velocities and to install formal structures to protect banks.

Present Control Practices

Soil erosion is controlled principally by limiting the amount of disturbance of prevailing conditions, limiting the time of exposure, controlling the amount of runoff and streamflow, and employing selected conservation and construction practices.

The natural cover of the ground serves to break the force of rain drops and disperse them, and to slow water runoff. Grasses, trees, leaves and litter also hold and release water by evaporation and transpiration. Disturbing this cover increases the impact of rain striking the ground, causing fines in the soil to form a seal at the surface, and allowing less infiltration and more runoff.

The longer this condition is allowed to exist, the more the run-off concentrates, forms rills and gullies, and increases in erosive force. Control measures must anticipate this action, and contain erosion within reasonable limits or not allow it to start. These measures may be simply a matter of (1) establishing new ground cover, (2) grading to create diversions or settlement areas, (3) installing formal erosion or drainage structures; and then (4) providing necessary maintenance. Controlling stream erosion requires special precautions.

Ground cover. Protective cover may be provided by seeding or laying sod, spreading mulches such as straw or hay, planting vines, shrubs or trees, or placing erosion-resistant materials such as asphalt emulsion or gravel. Sod is preferred for immediate, permanent protection in critical locations; seeding is more economical if conditions allow waiting for a stand of grass or other growth to develop. Mulches are used as temporary or permanent measures and may be useful until grass or other cover is established. Vines, shrubs and trees are useful for their aesthetic qualities in landscaping as well as providing erosion resistance. Gravel, stones and asphalt are particularly suited to areas subject to runoff at scouring velocities.

Grading. Change in the land surface may be desirable in minimizing slopes to reduce the velocity of runoff, to form berms to channel water in non-critical directions, and to create swales, terraces and wide ditches to contain runoff and maximize infiltration. Detention basins required for flood prevention can serve a dual role and provide control of sedimentation.

Structures. Erosion-or drainage-control structures may be desirable in critical areas. Sediment dikes and dams, debris basins, and special conveyance facilities may be used. Dams may be only ditch checks to interrupt the flow of water and allow a reduction in grades

and lengths of slopes. They may also be larger structures with spill-
ways or standpipes that are designed to intercept flows and allow set-
tlement of waterborne materials. In some locations, storm sewers,
paved spillways, or pipe drop chutes are necessary due to the amount
and velocity of water to be conveyed. Subsurface drainage may be used
to remove excess groundwater to improve vegetation and to reduce run-
off in critical areas.

Maintenance. Regular inspection and maintenance is necessary
in each of the above alternatives. Vegetation must be fertilized and
trimmed, and weed and pest control may be desirable. Sedimentation
and debris that tends to defeat the purpose of the control device should
be removed as needed. Concrete and asphalt should be inspected for
cracking and deterioration, and points of scour or bank failure should
be repaired. Project design must weigh the cost and likelihood of
adequate maintenance against the type and extent of preventive mea-
sures employed.

Stream erosion. Soil erosion in waterways is reduced by con-
trolling the velocity of flow and by creating a barrier to erosive action
within the limits of the waterway. Allowable velocities depend on
the type of soil, gradient of the waterway, and the measures used
to resist erosion. Energy dissipators may be used in outfall sections
and at points of constriction, and detention areas may be used to
meter flow through a channel to reduce velocities. Vegetation may be
sufficient to reduce the rate of flow and scouring action. Ditch checks
and paved or riprapped spillways allow abrupt changes in grade and
reduced slopes. Banks may be protected by sod, paved slopes, riprap,
rock cribs, groins, sheet piling, jetties and fencing.

Economics of Erosion and Sedimentation

The loss of soil by erosion is costly due to the decreased
value of the land and the increased cost of dealing with the sediment
produced. Agricultural productivity may be partly or completely des-
troyed depending on the type of soil and the degree of erosion. Develop-
ment costs are increased when it is necessary to restore the land
surface for construction and to establish desirable ground cover. Re-
moving sediment from water bodies and water supplies is also costly.
The cost of removing sediment and correcting problems is borne, in
many cases, by the public and not by those who benefit from culti-
vated or developed land.

Sediment damage in the United States was estimated at $262
million in 1966, as follows:

Deposition on flood plains, $50 million; storage space destroyed in reservoirs, $50 million; dredging sediment from inland navigation channels and harbors, $83 million; removal of sediments from drainage ditches and irrigation canals, $34 million; other damages including sediment removal, cleaning and added maintenance, $31 million;[3] removal of excess turbidity from public water supplies, $14 million. Cleaning sediment basins and water reservoirs is usually more costly than building them. The Soil Conservation Service (SCS) of the United States Department of Agriculture estimated in 1967 that new public storage capacity costs from 10 to 30 cents a cubic yards, and removing sediment costs from 25 cents to $1 to $2 a cubic yard.[4]

The cost for control on a highway with an average construction cost of $1 million per mile has been estimated at between $10,000 and $15,000 per mile.[1] The cost of control in housing developments has been given as $40 per lot by engineering and geologic consultants and $100 per lot by land developers.

The Soil Conservation Service in conjunction with the Illinois Department of Business and Economic Development is presently studying erosion and sedimentation in each county in Illinois and some of their early results illustrate the magnitude of the problem in the C-SELM area. One aspect of the study is a determination of the amount of soil loss by erosion on rural land undergoing development. Soil types and slopes were evaluated in representative areas in the rural portion of each county and a weighted average developed. Soil loss in Cook, DuPage, Lake, and Will counties was estimated at 35.1, 37.8, 40.6, and 31.8 tons per acre per year respectively.[5]

Using data on the projected loss of agricultural and vacant land between 1970 and 1975[6] and deducting the proposed development of regional open space, a total of about 15,000 acres of rural land in the four counties can be considered to be disturbed every year. Much of this is in the C-SELM area. Using the above rates by county results in an estimated 540,000 tons of soil lost per year due to development. Similar data is not available for Indiana.

A second part of the study was an estimated damage due to erosion in the remaining rural area in each county. Data from a 1967 survey[8] was used to (1) determine the total rural acreage, (2) that portion of the total that needed some erosion control treatment, (3) the annual soil loss without treatment, (4) the annual damage to agricultural productivity and to waterbodies due to sedimentation, and (5) the cost of appropriate treatment such as contouring, terracing, and grassing waterways, not including the cost of drainage. Costs are as follows:

County	Annual Soil Loss (million tons)	Annual Damage (million $)	Capital Cost of Treatment (million $)	Estimated Annual Cost of Treatment (million $)
Cook	0.885	2.48	5.91	0.52
DuPage	0.552	1.46	3.64	0.32
Lake	0.807	2.26	5.70	0.50
Will	1.878	5.26	13.67	1.19

Losses due to sediment were estimated to be much less than land productivity losses. In Cook County for example, damage due to sediment in ditches, streams and lakes was estimated to be only $270,000 of the total per year. Conversion of the capital cost of treatment to an annual cost might be made on the basis of a 10 year life for grassed waterways, a 20 year life for terracing, diking and similar improvements, and a 50 year life for structures. If a weighted life of 20 years at 6% is assumed, annual costs of $0.52, $0.32, $0.50 and $1.19 million for Cook, DuPage, Lake, and Will counties would result as shown above.

Sedimentation also causes damage by filling flood storage capacity in streams and floodways and increasing flooding. Sediment fills stream beds and detention basins, and causes flood water to build up at bridges and channel restrictions and overrun stream banks and flood plains. Damage results from the increased flooding and deposition of overwash materials on flooded areas.

Programs and Regulations

The Soil Conservation Service is most influential in this work, with a long history of involvement in agricultural areas. They have also been active in the urbanizing areas in recent years, providing information and counsel to local units of government and others. The Environmental Protection Agency and the Department of Transportation have recently become more involved in erosion control.

The Soil Conservation Service functions primarily to provide information on erosion and sedimentation, and influences practice only by voluntary action of developers and landowners. In addition to publications and personal assistance on good practice and techniques, detailed soil surveys are available in some areas that classify soil types, slopes, and erosion, and list characteristics and limitations of

the soils for different purposes. In the C-SELM area, complete soil mapping is available in Grundy, Kendall, Lake and Will counties in Illinois, and Lake County in Indiana. Partial mapping is available in the remaining counties.

The Federal Highway Administration of the Department of Transportation requires "prevention, control, and abatement of water pollution resulting from soil erosion" on all Federal and Federal-Aid highway construction.[8] This requires the contractor to provide necessary temporary soil erosion measures at the discretion of the resident engineer in addition to permanent features included in design. This may be modified and detailed by the States to fit their particular circumstances. The States may, and often do, apply these or similar requirements on other roads that are not Federally aided.

The Environmental Protection Agency (EPA) is interested principally in the effect of sediment on water quality, and in encouraging local units of government to require developers to take steps to control erosion. A new publication, "Guidelines for Control of Erosion and Sediment Deposition", was recently published and EPA's activity in this area of interest is increasing.

There has apparently been little or no local governmental control of construction practice for erosion and sedimentation purposes in the C-SELM area in the past. It is accepted practice in other parts of the country that have more severe problems. Maryland passed a sediment control law in 1970 and all its counties have since adopted suitable ordinances. Local soil conservation districts such as in Morris County, New Jersey have prepared extensive standards and specifications that may be adopted by local units of government.

Lake County, Illinois is the only local body currently considering an erosion control ordinance for new development, to be administered by the County or the local Soil and Water Conservation District. A comprehensive program is proposed with requirements based on guidelines prepared by SCS and modified to fit local conditions.[9]

Recommendations

A broad soil conservation program should be implemented in the C-SELM area. Developing land creates the most severe erosion and resulting sedimentation and should receive the greatest attention. The following soil conservation practices were recently proposed by an SCS official and are worthy of mention:

"1. Save natural vegetation such as trees, shrubs,
and sod whenever possible. Maintain the
natural beauty of the area.

2. Avoid unnecessary disturbance of soil.

3. Install roads and storm drains early in a sub-
division development.

4. Stockpile the topsoil for revegetating critically
eroding areas.

5. Minimize the time soils are left bare and vul-
nerable to accelerated water and wind erosion.

6. Use temporary seeding or mulching on denuded
soils during construction.

7. Limit the steepness and length of cut-and-fill
slopes, both to avoid the concentration of runoff
water and to facilitate establishing and maintain-
ing good vegetative cover.

8. Make provisions to accommodate runoff, especially
the increased runoff caused by changed soil and
surface conditions during and after any land dis-
turbance. This should be done without causing
erosion or flooding on adjoining lands.

9. Use basins to trap sediment on the site until the
disturbed area is stabilized with vegetation. These
sediment basins, diversions, etc., should be con-
structed before disturbing the major land area.

10. Engineering to take care of increased runoff or poor
drainage. New buildings and streets change the
old relationship between soils and vegetative cover.
Soils that were well drained under crops or native
plants may become poorly drained, or even unstable,
if a high percentage is compacted, covered with
buildings, streets, or parking lots."[10]/

A guiding principle should be to prevent sediment from leaving
the site. The most effective way to accomplish this is to prepare and
implement a comprehensive erosion control plan, administered and en-
forced by a responsible control agency. Permanent and as many tem-
porary erosion control measures as possible should be incorporated in
project design, and not be included as an afterthought when development

is underway. The design should include such features as temporary haul roads, borrow pits, and equipment storage sites that may not normally be included.

Soil types and slopes should be evaluated and preventive measures provided as needed. Dikes or sediment basins are not necessary where slopes are moderate and soils relatively stable, and no precautions may be necessary if the site is left uncovered for a short period of time. Table B-IV-I-1 lists the soil groups that have potential for erosion and sedimentation.[11] On sites where a soil survey is not available, slopes can be inventoried and soils sampled and classified in those areas only.

A model ordinance should be developed based on guidelines from SCS and EPA and a concerted effort made to have regulations adopted by counties and municipalities in the region. Some communities require that a developer advise them of site conditions and proposed conservation practices so that they can evaluate the proposal. Information could include soil type, slope, and degree of erosion, proposed elevations and improvements, and flooding of record, and could be a condition of receiving a permit to develop the site. This information should be mapped, and could be supplemented by a checklist similar to the one shown in Figure B-IV-I-1, which is required in Kalamazoo, Michigan. [12]

Requirements for Federal and Federal Aid Highways should be extended to other street and highway construction. It is estimated that 470,000 miles of secondary and rural roads are in need of erosion control improvements, ranging from $275 to $15,000 per mile, with an additional $50 per mile per year required for maintenance.[1] Erosion on most non-Federally aided road construction can be covered by acceptance of control practices by counties and the toll authorities.

The importance of erosion control on highway and other public works projects would be emphasized if required on environmental impact statements. Potential problems of a temporary nature may be overlooked in the evaluation of a project, and in the case of erosion and sedimentation, this can be the most critical period.

Rural areas should continue to employ conservation practices to control erosion on agricultural and urbanizing land. The federal cost sharing program for improvements now allows expenditures of up to $2,500 a year per farm, and participates on a sliding scale depending on the type of improvements made.

Table B-IV-I-1

SOIL GROUPS WITH EROSION AND SILTATION LIMITATIONS
Source: SCS Soil Interpretation Guide for Urbanizing Areas[12]

Group	Sub-Group	Soil Description	Soil Limitation
G2	1	Well and moderately well drained soils on 0 to 7 percent slopes and underlying materials; on uplands.	Erosion and siltation likely during construction and lawn establishment.
G2	2	Well drained soils on 2 to 7 percent slopes that have loamy sub-soils and sand and/or gravel at 2 to 5 feet; on uplands and terraces.	Erosion and siltation likely during construction and lawn establishment.
Y1	1	Well drained soils on 7 to 12 percent slopes that have loamy sub-soils and sand and/or gravel at 2 to 5 feet; on uplands and terraces.	Erosion and siltation during construction and lawn establishment.
Y1	2	Well or moderately well drained soils on 7 to 12 percent slopes that have loamy subsoils and underlying material; on uplands.	Erosion and siltation during construction and lawn establishment.
Y1	3	Well to excessively drained soils on 4 to 12 percent slopes that have loamy, sandy, or gravely subsoils and sand and/or gravel at less than 2 feet; on uplands and terraces.	Erosion and siltation during construction and lawn establishment.

Group	Sub-Group	Soil Description	Soil Limitation
Y2	1	Well to moderately well drained soils on 0 to 7 per cent slopes that have clayey to loamy subsoils and underlying material; on uplands.	Erosion and siltation during construction and lawn establishment.
Y2	3	Well to somewhat poorly drained soils on 0 to 12 percent slopes that have silty (silt or silt loam) or dense loamy (fragipan) subsoils and silty or loamy underlying material; on uplands and terraces.	Grading and excavation exposes highly erodible material.
Y3	1	Well to moderately well drained soils on 7 to 12 percent slopes that have clayey to loamy subsoils and underlying material; on uplands.	Erosion and siltation during construction and lawn establishment.
R1	1	All well drained and moderately well drained soils on slopes exceeding 12 percent that have sandy, loamy, clayey or silty subsoils and underlying material on uplands and terraces.	Severe erosion and siltation during construction.
R1	2	Well drained to somewhat poorly drained soils on 0 to 7 percent slopes that have hard bedrock at depths less than 3 feet; on uplands.	Erosion and siltation hazard.
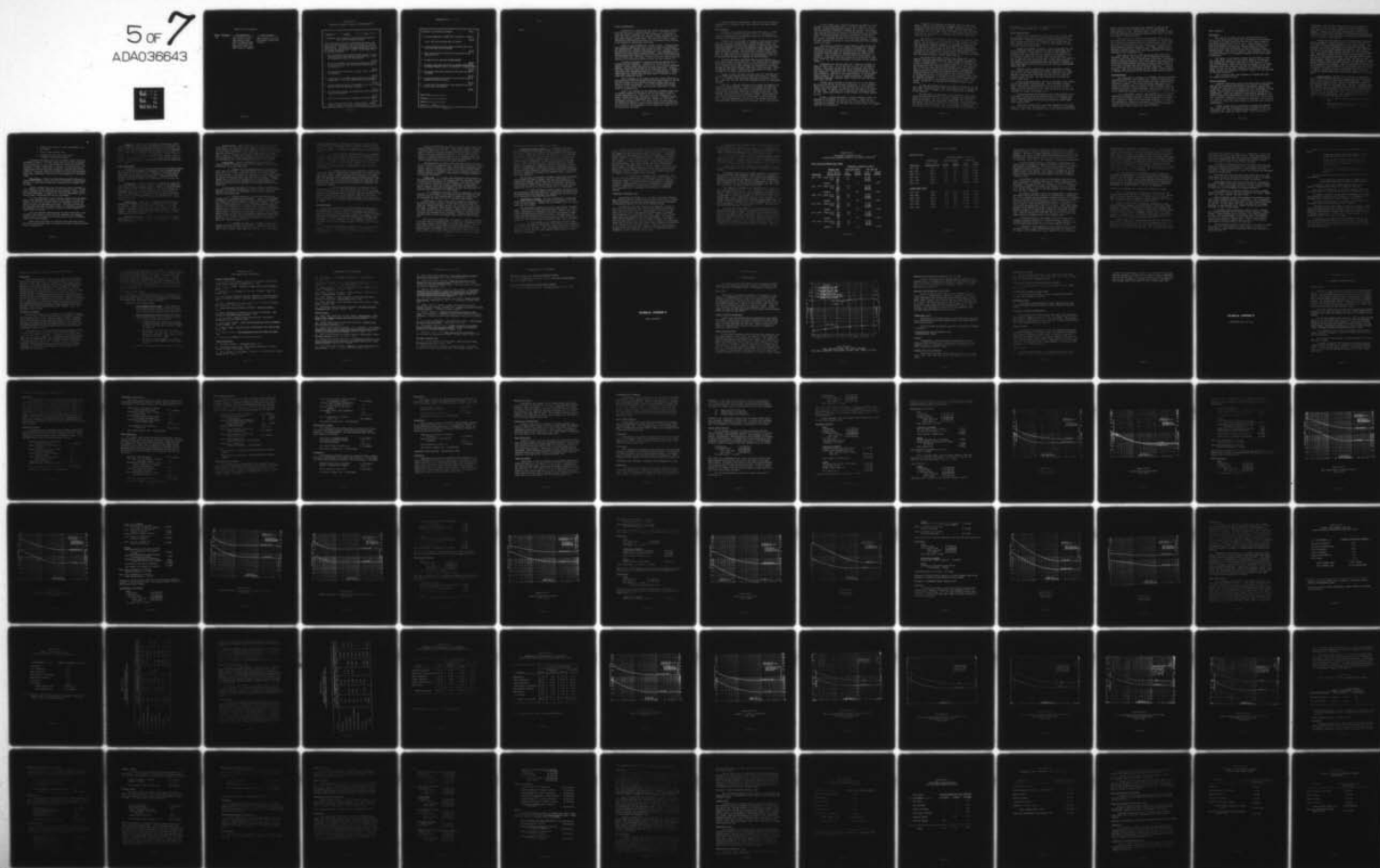
Table B-IV-I-1 (Continued)

Group	Sub-Group	Soil Description	Soil Limitation
R2	1	Well drained to some-what poorly drained soils on slopes exceeding 7 percent that have bedrock at depths less than 5 feet; or. uplands.	Severe hazard of erosion and siltation during construction.

KALAMAZOO EROSION CONTROL QUESTIONNAIRE[13]/

To_____   _____
   Supervisor   Township   Plat

INFORMATION TO BE SUBMITTED BY DEVELOPER WITH PRELIMINARY PLAT PLANS FOR TENTATIVE APPROVAL

Since considerable soil erosion and sedimentation take place during construction and development of a plat, it is necessary to plan some control measures. The measures should apply to all features of the construction site, including streets, utilities installations, as well as the protection of individual lots. Therefore, will you please fill out the following questionairre.

1. Has the Kalamazoo Soil Conservation District and the U. S. Soil Conservation Service been contacted for soil information, erosion and sedimentation standards and specifications?

   ___ ___
   Yes No

2. Do you have watershed, soil erosion and drainage information for the entire tract of land with a boundary line showing the plat to be developed first?

   ___ ___
   Yes No

3. Does the plan show where grading, stripping, cutting, or filling is contemplated?

   ___ ___
   Yes No

4. Are temporary and/or permanent seedings being used on graded or stripped areas? If yes, what seeding mixtures are being used?

   ___ ___
   Yes No

5. Does the development plan fit the topography in such a way to create the least erosion and sedimentation potentials?

   ___ ___
   Yes No

6. Are natural grass waterways and drainage ways being protected and shown on the plat plan?

   ___ ___
   Yes No

7. Are there any constructed grass waterways or diversions planned?

   ___ ___
   Yes No

   If yes, give width, depth, grade, and drainage area - seeding materials - fertilizer to be used - mulching materials to be used.

Information to be Submitted by Developer   Page 2

8. Are there slopes that are steeper than 2 horizontal to 1 vertical?

                                                              ___ ___
                                                              Yes No

    If yes, what kind of retaining walls are planned?

9. Do plat restrictions provide for grading of building sites so that
    water flows away from the buildings?

                                                              ___ ___
                                                              Yes No

10. What mulching material is being used on side slopes that are
    steeper than 4-1?

11. Is there any tile or open ditch drainage planned?

                                                              ___ ___
                                                              Yes No

12. Is excess surface water from the plat or watershed being diverted
    and properly disposed of either on a temporary or a permanent basis?

                                                              ___ ___
                                                              Yes No

13. Are temporary debris basins planned and used during the construc-
    tion period?

                                                              ___ ___
                                                              Yes No

14. Are permanent leaching basins shown on the plan with sodding or
    seeding to stabilize side slopes?

                                                              ___ ___
                                                              Yes No

15. Is proper water and sedimentation control provided for during the
    various stages of development?

                                                              ___ ___
                                                              Yes No

    Name of Plat_____

    Developer_____

    Engineer_____

    Section_____ County_____

Figure 9-IV

WATER CONSERVATION

Conservation of potable water has become increasingly important in recent years due to increased demands for it. Per capita use of water has steadily increased, and the concentration of population in urban areas has created and will continue to create shortages of available water supplies in these areas. The extent and nature of future conservation requirements and practices is an important policy question to be answered in metropolitan areas today.

Water use has increased in a number of ways. Increased industrial production requires more water, and new industrial technologies, like petrochemicals, demand large amounts of water. Personal affluence has resulted in an increase in swimming pools, air conditioners, automobiles, and water-using household fixtures. Where washing dishes by hand previously used about three gallons of water a day, dishwashers now use six gallons a day. Previously, no water was used to dispose of garbage: disposals now use about two gallons of water a day per household. [13]

A number of methods for conserving water are possible and are practical here and elsewhere. Industrial and commercial use is restricted by water charges and controls on its use in diluting wastewater effluent. Recycling and reclamation are often economical. Leakage from water mains and other parts of the distribution system is limited by construction and maintenance programs on a continuing basis. Wastewater, rather than potable water, has been used for irrigation in many areas with satisfactory results.

Water shortages in the C-SELM area have caused difficulties in allocating water from Lake Michigan and other prime sources in the past and will continue to be of concern although shortages here are not nearly as serious as elsewhere. One of the largest single demands on water in the C-SELM area, and a promising candidate for conservation is residential use.

Data is not available for the proportion of domestic water used for different purposes in the region, but the breakdown in another, smaller metropolitan area, Akron, Ohio, may be representative. In Akron, domestic water use is divided as follows: cleaning (washing, bathing, etc.) 45%, toilet flushing 41%, cooking 6%, drinking 5%, and garden watering 3%.[14] In a recent survey, in-house water use as estimated by plumbing contractors and manufacturers averaged about 45% for toilets, 28% for bathing, 23% for laundry and dishes, and 4% for drinking and cooking.[15]

Only two means of conservation, affecting the bulk of domestic consumption, are discussed here: water metering, and water saving devices.

Water Metering

Metering is an accepted practice across the country. A 1969 report of the Water Industries Division of the U. S. Department of Commerce showed that 88% of residential users and all industrial and commercial users were metered.[16]

Metering has the advantage of allowing a more equitable assessment for water used, reducing flow to sewage treatment plants, and achieving economies in water supply systems by delaying water treatment plant expansion and saving on chemicals, power, and other operational requirements. It has the disadvantage of added expense for meter installation, reading, and maintenance, increased billing costs based on usage rather than a flat rate, and the loss of consumer surplus. It might also have an influence on the appearance of the city in reducing lawn watering and cleaning of exterior surfaces.

In only a few areas, principally New York and Chicago, are there large numbers of unmetered users. Full metering was recommended in the past in New York City, where only about one-quarter of the service is metered, due to water shortages and opposition to increasing supplies by constructing additional reservoirs in the mountains. Universal metering and repair of system leaks was expected to reduce consumption by 150 million gallons per day.[17]

Studies of water use before and after meters are installed have usually shown more judicious use of water and decreases in consumption. It is generally held that water saved does not present a hardship to the user, but only limits water wasted and used for nonessential purposes.

The lack of metering contributes to the liberal and wasteful use of water for many purposes. A public more conscious of water use could be expected to limit the times cars are washed and make more use of public car washes, where use is metered and paid for. More emphasis conceivably would be placed on filtering and retaining water in pools, and they would not be drained as often. Lawns, which in many cases are watered more than recommended, would be sprinkled less. Increased awareness of water costs to the resident would also encourage fixing leaking fixtures and restricting use for other purposes.

In the C-SELM area, virtually all services are metered with the exception of most residential users in the City of Chicago. All commercial, industrial and apartment buildings (all "3-flats" and above) and some individual residences in Chicago are metered. And although many apartment buildings are metered, individuals within the apartments may not be directly metered and are not constrained in their use of water. It is estimated that about one-third of the users in the City are metered, accounting for about 55% of the water supplied. Charges for water elsewhere are based on a flat rate determined by the assessed value of the home.

The City has a program to install meters in areas as they are redeveloped, and meters will be installed if requested, but complete metering would be an enormous and expensive undertaking. Meters are usually installed inside the building but may be added at house connections near the street in a weatherproofed vault when entry is a problem. Vault installations cost over $200 for each service. Installation, maintenance, and meter reading would be difficult in some areas of the city, and would undoubtedly be resisted by many property owners.

Philadelphia was the last large city to institute a program to meter all water users. Of 515,000 users in 1952, 138,000 were unmetered. These water services were all in residences built before 1918 when a building ordinance was adopted requiring meters in all new construction. Starting in 1966, for a period of five years, meters were installed in almost all of the remaining homes, at the property owner's expense. The cost of a standard installation, including a preliminary inspection, overhead, meter, fittings and labor was about $35 at that time.[18]/

Average figures indicated a decrease from 677 to 372 gallons per service per day for the unmetered residences, or a 45% reduction. Some of this was attributed to slum clearance and a leak reduction program during this period, and possibly some was due to an increase in water rates. A reduction due to metering was estimated at about 28 1/2%, which would amount to a total savings of 27 million gallons per day.

Boulder, Colorado experienced a similar reduction in water use following the installation of meters in all homes. Annual use was estimated at 243 gallons per capita per day in 1960 when 5% of the community was metered. In 1965 after 100% metering, water use dropped about 40% to 149 gpcd.[19]/

A study of the feasibility of installing meters in Idaho Falls, Idaho in 1959 by a consulting firm estimated a reduction of about 50% in daily use and over 60% in peak use.[20] Observed decreases in Plattsburg, New York, Walla Walla, Washington, and other areas were in the 40% to 50% range.[21] Urquhart's "Civil Engineering Handbook" notes that the installation of meters has been shown to "decrease usage by 30 to 40%."

The effect of 100% metering in the C-SELM area cannot be determined with precision due to the influence of system leakage and other factors. As an illustration, however, if an average saving of 193 gallons per day per unmetered household (as estimated in Philadelphia after metering) is used for 355,000 households, savings could amount to a total of 68 million gallons per day. From another standpoint, if a savings of 30% of normal per capita domestic consumption of 80 gallons per day is assumed – for 355,000 services at 3 persons per service – the total would be about 25 million gallons per day.

This would reduce water treatment costs slightly due to lower chemical and power requirements and reduce the amount of sewage processed. Assuming that 60 to 70% of water supplied for domestic use results in sewage effluent, it is possible that from 15 to 45 MGD could be removed from flows entering treatment plants. If it was assumed that all flow was taken from the West-Southwest Plant, a reduction of from 1.5 to 4.5% of design flow could be realized. Although this would increase the strength of the sewage to be treated, economies would be possible due to the decreased volume to be processed. It is assumed that the cost of the collection and conveyance system would be relatively unchanged. A savings of about $3 to $10 million annually would be possible for the advanced treatment plant system, if all capital, replacement, and operating and maintenance costs are considered.

For the land treatment system, the capital cost would be affected little since the pollution load is the most important factor in design, not the volume. Only a savings of $0.6 to $2 million would be possible in this instance.

The installation of water meters would undoubtedly reduce water use in presently unmetered areas and the advantages and disadvantages should be evaluated. The cost of metering includes the cost of the meter and necessary leads and fittings, installation, meter reading and maintenance, and increased billing costs. The City of Chicago estimates the cost of universal metering at $65 to $80 million. Meter reading is estimated at $4.60 per year, and maintenance (based on figures from

Philadelphia) at $0.40 per year. A comparable total annual cost would, therefore be about $6 to $7 million.

Water Saving Devices

Domestic water conservation using "water saver" toilets, washing machines, and shower heads, and reusing wastewater is also a possibility.[15] Although the changes in facilities resulted in a reduction of potable water use of 32%, savings to the property owner in water charges were found to be too small to justify making a change in fixtures based on normal rates.

Savings are possible, of course, if water saving units are used in new construction and in replacing old units in rehabilitation and renovation efforts. Toilets that use 3.25 gallons when flushed, as contrasted with those using 5.25 to 8 gallons, are available and can be included in building and plumbing codes and specifications. A cost of about $140 was used for the water saving model in the analysis, only slightly more than the standard unit. A local study indicated that replacing toilets installed before 1929 that require 7.5 gallons per flush might reduce water use by 50 mgd in the six county Northeastern Illinois area.[22]

Washing machine manufacturers can be encouraged to design units similar to some machines now available (requiring 32 to 42 gallons for an 8 lbs. load, respectively) that use about 30% less water than the average unit and about 45% less than the units using the greatest amounts of water. The water saving units cost about the same as other units.

Water saving shower heads can also be included in new construction and improvement of existing units. The replacement of shower heads came closest to being justified on water saved on an annual basis. The flow regulator head cuts water used in half, or up to an estimated 9% of total in-house use. A cost of $8.75 for the head was compared to a regular unit costing $6.15.

Promotional efforts and marketing of the preferred models by the municipality or water utility may prove beneficial, and financing and home improvement loans can be directed to their use. Reduced water rates or other inducements could be made available for those using water saving devices.

Total water savings would vary greatly depending on the acceptance and use of water saving facilities. To illustrate the possibilities, however, if half of the new dwelling units in the region used the above

devices, and only 1% of existing units converted to them per year, there could be a savings in water every year of approximately 5 million gallons per day. This would mean a further reduction of about $1 million annually for the advanced treatment plant system, and about $0.2 million for the land system.

The illustration of a small sewage treatment facility on-site that allows reuse of treated water for toilet flushing and garden watering is directed at households that would ordinarily use a septic system. The study concluded that even with a number of homes using a combined system and achieving economies of treatment, widespread use would not be feasible for a number of years. The decreasing use of individual treatment systems also means the application is of reduced importance in the metropolitan area.

Some form of on-site or local processing would seem to be worthwhile, however, that could be used in conjunction with public sewers to make limited use of treated water in the home. For example, separate plumbing connections could allow relatively unpolluted wastewater from washing machines and other relatively "closed systems" to be directed to holding tanks and treatment facilities and not allow it to be mixed with other more heavily polluted, wastewater. Collected stormwater might also be integrated into the system. Further research to evaluate the possibilities of this general concept would be helpful.

Recommendations

The conservation of supplies is a worthwhile pursuit and deserves considerable attention in the C-SELM area. Efforts to reduce domestic water use has not kept pace with industrial and other conservation and reclamation programs. Water meters and water saving devices are effective in controlling water use without creating a hardship, but are expensive to place in existing dwellings. This investigation does not clearly indicate that a broad conservation program solely to reduce the cost of wastewater treatment is justified and therefore it is not recommended.

It is recommended, however, that metering be used in new and rehabilitated residences, and that further study be given to the possibility of more widespread adoption of water saving devices in residential, commercial, industrial and institutional buildings. It is also recommended that research and development of individual, on-site systems for wastewater treatment give full consideration to the reuse of treated water to further reduce total water utilization.

SEPTIC SYSTEMS

Introduction

The wide use of septic tanks and absorption fields in the past in areas where public sewers were not available, or were deemed too expensive to install, means that they will be a major concern in wastewater management for years to come. They continue to fill a need in developing areas and can, under acceptable conditions and with proper design and maintenance, provide satisfactory on-site sewage treatment under most conditions without detrimental effects to the environment. They may be, therefore, more desirable or at least a suitable alternative to a community wastewater system under certain circumstances.

The number of new systems installed has decreased in recent years with tighter controls and with greater emphasis on comprehensive planning and furnishing a full range of public services at the time of development. Septic systems that are installed now are better controlled, have better materials, and have the benefit of more information on local soil and water conditions and good design practice. Further advances and uniformity of practice, however, are still desirable for functional optimization.

This discussion deals with strategies for dealing with existing systems and future systems.

Present Conditions

Experience with septic systems and recent research and development has led to improved specifications and practices, and systems installed today have greater assurance of satisfactory operation than in the past. Knowledge of the importance of soil, groundwater, on-site testing, system layout, and maintenance has led to more effective requirements. Codes and ordinances covering septic systems are frequently developed and administered by local units of government, however, and lack in uniformity and up-to-date provisions. The net result is a variation in policies and standards for construction of new systems depending on locale, and septic systems in place that range greatly in effectiveness.

Septic systems have the advantage of needing little attention and, if good practice is employed, provide an efficient system at a reasonable cost. They are frequently more economical on large lots where good soil exists than public sewers. They are attractive to

a developer of such locations since they can provide satisfactory treatment as homes are built. Building and paying initially for a large system for part of the eventual service area is therefore not required. Properly operating septic systems do not produce point discharges nor should they contribute to eutrophication through nutrient input to a stream or lake since they make use of the natural qualities of soil to purify the septic effluent.

Present-day septic systems have the disadvantage of requiring regular maintenance to function properly, of a limited life and of a performance that is seriously affected by random storm events. They are subject to problems depending on the care taken when they are installed, and to increased groundwater elevations. The effluent from the bank is foul and when effluent discharges into water bodies or runs on the ground without suitable treatment it can be very objection-able. Since septic systems are not point sources of effluent, it is difficult to apply advanced treatment. The homeowner must be selec-tive in what is disposed of in septic systems, but need not be in public systems. They are not perceived to be as dependable as sewers, and the values of homes with them are lower, possibly $500 to $1,000 where alternatives are available. There is presumably less control over the installation to assure adherence to good practice than with communi-ty systems, and there is no way of monitoring the effluent to see that it conforms to acceptable levels. Whether they create as serious a problem as treatment plants when not operating properly is a moot ques-tion.

Existing systems. Systems now in use that are ineffective and damaging to public health and the environment may not have been maintained correctly, may have been installed improperly, or should not have been installed at all. Almost all problems are caused by the absorption field, not the septic tank. The tank, if properly sized and designed, performs a relatively simple operation, separating solids from the wastewater generated so that they do not enter and clog the filter materials in the absorption field. The tank is a closed system, protected from the elements, that only requires occasional removal of sludge to operate satisfactorily. The absorption field, on the other hand, is critical in several situations when:

1. The soil will not allow percolation at an adequate rate,

2. Groundwater inhibits percolation and retains effluent just below the surface of the ground, allows it to move laterally, or forces it up.

3. Surface water floods the trench and interferes with percolation,

4. Filter or pipe materials clog,

5. Trenches are too close to each other or to wells or surface water bodies.

Inadequate percolation can result in polluted surface water and groundwater supplies. Too many septic fields in an area can saturate the underlying soil and not allow adequate percolation and treatment. Individual cleaning or replacement of laterals may be necessary or the installation of public sewers may be required. Laterals may become clogged with roots and mechanical cleaning or chemical action may be necessary. Excessive sludge in the field due to poor tank maintenance means early replacement of the field or the installation of a new field nearby.

New systems. Systems presently being installed make use of research and experience but do not take full advantage of technological advances, so that basic concepts and components have changed little for years.

Systems installed today have the benefit of better knowledge of soil conditions and their limitations for use with filter fields. In some soils, limitations can be overcome with good planning, careful design, and sound management. In other soils, with poor permeability and excessive water, overcoming them may not be feasible.

It is now understood by most regulatory agencies that percolation tests are not infallible, that different soil types should be tested differently, and that results depend heavily on techniques, type of soil, time of the year and knowledge of the area. How the test holes are dug and filled with water can affect results, and conditions of wetness not apparent at the time of testing can render a system inoperable at certain times of the year.

It is also generally understood that the layout and extent of the absorption field should be varied with site conditions and proposed loading, and that the systems are desirable only in areas of relatively large lots and low densities.

Septic tanks used today are generally larger than those used in the past so that detention time is increased and reserve capacity is available for changing use.

Regulation. Control of septic systems is provided by building codes or ordinances in most areas, and inspection of an installation is usually required by qualified personnel before it is put into use. The county is the basic governmental unit for control of septic installations in the C-SELM area, covering all unincorporated areas and some or all municipalities. In some counties, all municipalities follow county regulations, in others, local units of government have separate provisions. Published specifications are available in all counties except LaPorte in Indiana. Building permits are typically not issued until the septic system is approved.

Future Possibilities

The options open for dealing with existing and new septic systems are an important consideration in developing a total wastewater management program in the C-SELM area. Strategies for maintaining and rehabilitating existing septic systems, and connecting to public sewer systems should be formulated and evaluated. Improvements for new systems should also be studied and incorporated into regulations and practice.

Maintenance. Periodic maintenance is essential to proper functioning and long life of a septic system. Clogging of the absorption field, probably the most frequent cause of trouble, is caused by solids, that should settle out, passing through the tank. This is caused by a retention time that is too short (when the tank is too small), or a level of sludge that is too high (when the tank should be cleaned). There is no recommended maintenance for the absorption field, except for removal of roots. Flushing the laterals does not improve, and may worsen, conditions.

Rehabilitation. Repair involves digging up the tile field and replacing it, extending the laterals, or adding new laterals if space is available. Studies have shown that the performance of a field is improved if loading is stopped for a short period of time and then resumed. Alternating from one set of laterals to another is also advantageous. Replacing an undersized tank with a larger one, or adding a second tank may be necessary.

Central sewer system. Connection to a central system may be preferable when this is feasible. This is frequently no more expensive than rehabilitation on a short term basis, and is usually most desirable if one is available.

Aerobic system. Another possibility not included above is the replacement of the septic tank with an aerobic system which aerates the wastewater and achieves a higher degree of treatment and removal of solids before discharging into the drain field. From 30,000 to 50,000 units were estimated to be in use in the United States in a recent study. They are somewhat more expensive than septic tanks and have the disadvantages of being partly exposed to view and of requiring more maintenance due to the mechanical operation of the process.

Improvements. Several improvements in the processing of domestic wastes in individual units may be possible by research and development in the following areas:

a. Effluent from the absorption field is presently not given advanced treatment. Some experimental use of chlorination has taken place, and it may be desirable to make a practice of adding it to septic effluent. Flocculants have been shown to significantly increase the amount of suspended solids that settle out in the tank. Filtration through the soil can provide adequate removal of phosphates. Nitrates which can contribute to the eutrophication of receiving waters are partially destroyed with the remainder of the nitrogen available for fertilizer use.

It is estimated that about 50 pounds of nitrogen is produced per year from the wastewater produced by a family of five. This is enough to fertilize a one-quarter acre residential lot if it can be distributed uniformly and economically.

b. Adequate treatment is not provided when the absorption field is saturated with groundwater. Control of the discharge of effluent in rainy periods would be helpful, possibly by detaining it in a holding tank until satisfactory percolation is possible. This would require a pump controlled by a rain or groundwater-sensing device in addition to the holding tank, and would likely result in an increased operating and maintenance cost. Another method for dealing with possible periods of ground saturation would be to install a subsurface drainage system similar to those used for agricultural purposes. Such a system would maintain the ground in an unsaturated condition even during rainy periods, insure adequate percolation distances between tile field and drain pipe, and provide a control over the direction of movement of septic tank overflow through the soil.

c. Cleaning the septic tank at regular intervals is essential to the effectiveness of the system. A procedure can be instituted that makes an annual inspection mandatory by a responsible agency with cleaning a requirement when needed. The cost of inspec-

tion and cleaning could be added to local taxes, or the cost could be recovered by a reduction in taxes by providing evidence that maintenance is performed. Inspection and cleaning could be accomplished by a public agency, or by a private contractor with a low-cost service contract.

d. Local codes and regulations should be strengthened to include latest developments in the field, and to determine the desirability of septic fields where community sewers are possible or are planned. Maintenance provisions are generally lacking in local codes and should be covered. And since some problems with improperly installed systems are obvious in a short period of time, the use of performance bonds would assist in safeguarding the homeowner and the community.

e. Concrete septic tanks are universally accepted and are usually selected, although brick, concrete block and metal are also used. New plastic materials and fiberglass would seem to be applicable here because of their lightweight ease of shipment and installation, and resistance to corrosion and infiltration and exfiltration. Perforated plastic pipe can also be used in place of the open-joint clay tile due to its ease of installation and resistance to settlement, disjointing and clogging.

f. Interest has been expressed in recycling some of the wastewater for use in toilet flushing and lawn watering. As discussed in the section on water conservation, separation of wastewater generated in the household would allow use (with some treatment) of the relatively unpolluted water for this purpose, continuing to treat the heavily polluted waste through the regular process. This would reduce the cost of water used, and the amount of wastewater entering the septic system.

Recommendations

It is apparent that this is a complex matter and that there are no strategies that will fit all situations. The need for advances in new installations is obvious, and it is recommended that increased research and development effort be given to this area. There are instances where the permanent use of individual systems is advantageous, and more emphasis should be given to this in the future. The distribution of effluent over a wide area with no point discharge to streams or groundwater avoids some undesirable aspects of conventional treatment systems.

A strategy for dealing with existing systems as well as systems installed in the near future is also important. Local conditions and community objectives can result in a number of possible approaches to the problem.

Several considerations are regional in nature and an agency with regional authority may be desirable. If the septic system is viewed as a temporary measure for use until a central sewer system is available, then the counties or the larger sanitary districts may be the logical bodies to deal with questions in this area. If the septic system is viewed as a permanent treatment process, to be maintained and rehabilitated or replaced in kind as needed, then the communities and the smaller sanitary districts may be the proper authorities.

Recommendations are listed to deal with three general questions: (1) the development of guidelines and a model code to govern new installations, (2) the development of criteria and standards to determine when existing systems are functioning properly and if they should be rehabilitated, replaced, or abandoned, and (3) the evaluation of existing and proposed systems to determine if they should be considered permanent or temporary installations.

Model code. a. A model code employing the latest thinking on the construction and care of new systems should be developed and made available to local authorities for consideration. The provisions listed in Data Annex B, Section IV-I would be appropriate for this code, or may be used as a basis for revising present codes and regulations. A state or regional code could be desirable, possibly with provisions for inspection and enforcement of its requirements.

b. Guidelines and criteria should be developed to evaluate proposals for new systems. They should consider site conditions, present and future use and desired quality of ground and surface water, and feasibility and timing of local or area-wide sewer systems.

Connecting to a central sewer system should be considered as an alternative course of action. In most areas that have large lots, homes are relatively expensive and the cost of sewers is a minor consideration. Furthermore, where ground or surface water quality is critical, septic systems may not be feasible. Bacteria from septic effluent have been found to travel up to 200 feet through soil, and much farther in surface and underground water movement. Distances specified from absorption fields to wells and critical surface waters usually consider this.

Also, the rate of construction will influence a decision on whether to provide sewers or individual units. A slow rate of development might not justify a public system within a reasonable time period. In areas where central sewers are indicated and can be made available in a reasonable time, development may be postponed until then, due to the double expense of the septic system and the sewer, and the increased cost (up to 3 times as much) of installing sewers later when dwellings,

lawns and streets and other utilities are in place.

Adequacy of existing systems. a. Guidelines and criteria can be developed to evaluate the adequacy of existing systems. They should consider site conditions, age of systems and effectiveness of treatment, present and future use and desired quality of ground and surface water, and feasibility and timing of local or area-wide sewer systems. The myriad of considerations that arise in situations with varying degrees of groundwater or surface water pollution, effecting one or a number of dwellings, with or without public water supply, requires a good deal of judgment and cannot be adequately discussed with all its ramifications. Suffice to say that unsafe levels of pollution in drinking water, high coliform counts in stormwater runoff and surfacing of septic effluent in residential areas should not be tolerated, and some combination of individual and community effort is needed to solve these problems.

The first step in developing a program to deal with this problem is to inventory the septic systems, their age, effectiveness, etc., evidences of pollution, and programs and costs for the provision of central sewers in these areas. From this an evaluation can be made to determine the status of existing systems, and to rank the deficient systems with respect to the need for rehabilitation or replacement.

b. State and federal sources of grants and loans should be investigated to see what relief may be afforded for communities and homeowners faced with expenditures for upgrading their private systems as an alternative to replacing them with sewers.

Permanancy of systems. The key consideration in determining if the septic system is considered temporary or permanent is the provision of central sewers - whether they are desirable, and if so, when they would be provided.

The preceding discussion has covered some important considerations. The life of the initial installation has averaged 10 to 12 years before rehabilitation or replacement in the past. On this basis, the cost of providing central sewers at the time of development is almost always the most economical course of action. Providing sewers at a later date can be expected to cost half again as much on an annual cost basis, and can be a great inconvenience while under construction.

a. New systems may be considered permanent if they are used on very large lots where there are no plans for the installation of a central sewer system, or where the system will not be provided for about 20 years in the future. The system should anticipate varied and heavy use over its life, and high standards should govern its installation and maintenance.

A system may be considered temporary for any intermediate time period. If the provision of central sewers is imminent, where monies are budgeted or where construction seems certain in two or three years, development may be postponed until they are available. If a planned system is likely in a somewhat longer period, other courses of action may be taken. A sewer collection system may be installed and temporary treatment facilities employed until it is possible to connect to the central system. This option assumes that the treatment facility will not have a detrimental effect on the environment, and that it will be phased out when sewers are available. Another course of action would be to provide capped connections to the future sewer at easily accessible locations. Standards for these septic systems should not be relaxed in case the sewer system is delayed.

b. In the case of existing systems, planning for sewers may affect the rehabilitation or replacement of septic systems - delaying this expense for a short time if a sewer program is being implemented. Again, a collection system may be built and connected to a temporary treatment facility if this is advantageous as an interim measure. Rehabilitation or replacement in kind may be appropriate on a continuing basis if conditions are satisfactory for individual systems.

PHOSPHATE DETERGENT BAN

The question of whether or not to require a phosphate detergent ban cannot be answered without considering the various sources of phosphorus in effluents and receiving waters together with the percentages of phosphorus acquired from each of the principal sources.

In municipal wastewater treatment plant effluents, perhaps 30 per cent of the phosphorus comes from human wastes such as feces, urine, and waste food disposal. The remaining 70 per cent of phosphorus has its source in the phosphate builders found in detergents used principally for the laundering of clothes. Other sources of phosphorus may cause deviations from these percentages. For example, where hexametaphosphate or other phosphorus compounds are used as corrosion and scale control chemicals in water supplies, the phosphorus added will be present in the same concentration, although not necessarily in the same form as in the water supply. This source can account for 2 to 20 per cent of the total phosphorus present in wastewater.[38] Surface runoff also contributes a small but not insignificant amount of phosphorus in those relatively urbanized areas.

In rural areas, the situation reverses itself with the majority of the total amount of phosphorus in receiving waters coming from surface runoff which includes phosphorus-rich water from the fertilized root zone as well as from animal wastes. Human wastes, including laundry waste, comprise the remaining source in rural surface water.

As of June 1971, sixteen states had adopted wastewater effluent phosphorus standards. In most cases, effluent concentration limits range from 0.1 to 2.0 milligrams per liter (mg/l), as phosphorus, with many established at 1.0 mg/l. Where concentrations are not specified percentage reduction requirements range from 80 to 95 per cent.

Neither an effluent nor a percentage-reduction standard actually limits the phosphorus load in terms of pounds of phosphorus discharged per day.

If phosphate detergents were banned completely, the net effect upon suburban and urban wastewater quality would be a reduction of perhaps 70 per cent in phosphorus. The average total phosphorus concentration in raw domestic wastewater is about 10 mg/l expressed as elemental phosphorus. Recent observed concentrations in the C-SELM area, reflecting a City of Chicago partial phosphorus ban, were lower than this amount, as shown in Table B-IV-I-2.[39]

The reduction of phosphorus to a concentration of 0.4 to 3.4 mg/l in the wastewater from the MSD plants has a negligible effect upon the growth of algal blooms. This statement can be explained as follows. The principal basic elements in algae include carbon, nitrogen and phosphorus, in the approximate ratios of 100:15:1 (C:N:P). The effluent from a typical sewage treatment plant in the C-SELM area, consisting largely of dry weather flow, contains approximately 40 mg/l of carbon, 20 mg/l of nitrogen, and 8 mg/l of phosphorus when there is no ban on phosphates, or a ratio of 100:50:20 (C:N:P). The ratio with a ban could be on the order of 100:50:5.

In either case, these effluents obviously contain considerably more phosphorus than is required for phosphorus to be the controlling nutrient in the growth of algae. Phosphorus must be further reduced by a factor of twenty or five, respectively, before a level is reached where an additional reduction in phosphorus would begin to affect the growth of algae. The amount of phosphorus present in effluent at this point would have reached 0.4 mg/l This is the lowest value achieved in Table B-IV-I-2 at the West and Southwest Side Plant. Thus, the phosphorus removal accomplished by the City of Chicago phosphorus

Table B-IV-I-2

PHOSPHORUS REMOVAL BY THE
METROPOLITAN SANITARY DISTRICT OF GREATER CHICAGO[39]

WEST AND SOUTHWEST SIDE PLANT

| | | | | Phosphorus Content as Total P | | | |
| | Sewage Flow | | Parts per million | | lbs. per day | | |
Month/Year	MGD, Million gallons per day		Raw Sewage	Treated Sewage	Raw Sewage	Treated Sewage
July, 1972	Inflow W.S.	564	5.1		24,000	
	S.W.	367	9.0		28,000	
		931			52,000	
	Outflow	895		1.3		9,700
Aug., 1972	Inflow W.S.	552	4.3		20,000	
	S.W.	462	7.6		29,000	
		1014			49,000	
	Outflow	946		0.4		3,200
Sept.,1972	Inflow W.S.	511	3.4		14,000	
	S.W.	395	3.9		13,000	
		906			27,000	
	Outflow	850		0.4		2,800
Oct.,1972	Inflow W.S.	488	4.1		17,000	
	S.W.	347	5.9		17,000	
		835			34,000	
	Outflow	803		0.7		4,700
Nov.,1972	Inflow W.S.	493	4.9		20,000	
	S.W.	365	7.0		21,000	
		858			41,000	
	Outflow	821		0.5		3,400
Dec.,1972	Inflow W.S.	458	5.2		20,000	
	S.W.	358	5.4		16,000	
		816			36,000	
	Outflow	788		0.5		3,300

CALUMET PLANT

| | Sewage Flow | Phosphorus Content as Total P | | | |
| | | Parts per million | | lbs. per day | |
Month/Year	MGD, Million gallons per day	Raw Sewage	Treated Sewage	Raw Sewage	Treated Sewage
July, 1972	185.8	9.8	3.4	15,000	5,300
Aug.,1972	212.1	7.1	2.9	13,000	4,900
Sept.,1972	199.2	6.4	1.9	11,000	3,200
Oct.,1972	201.5	8.8	2.1	15,000	3,500
Nov.,1972	235.3	4.1	1.7	8,000	3,300
Dec.,1972	218.5	5.6	1.2	10,000	2,200

NORTH SIDE PLANT

Month/Year	MGD, Million gallons per day	Raw Sewage	Treated Sewage	Raw Sewage	Treated Sewage
July, 1972	343.4	3.9	2.6	11,000	7,400
Aug.,1972	371.5	3.2	2.0	10,000	6,300
Sept.,1972	356.3	3.3	2.0	9,800	6,000
Oct.,1972	331.2	3.7	2.4	10,000	6,600
Nov.,1972	352.2	3.9	2.1	11,000	6,200
Dec.,1972	327.8	3.6	2.1	9,800	5,700

ban appears to be effective in producing a treated effluent in which the residual phosphorus content is beginning to control the algal growth potential. Therefore, if further phosphorus removal was or could be contemplated, the amount of algal growth in the receiving watercourse could be diminished. Since this is, in fact, not the case, the net result of the phosphorus ban is little if any reduction in watercourse eutrophication.

To achieve further reduction of phosphorus in wastewater, provisions must be instituted at wastewater treatment plants for the additional removal of phosphorus. This is usually accomplished by chemical precipitation through the addition of lime or mineral salt. Materials commonly added to wastewater for the removal of phosphorus are lime, ferrous sulfate, ferric chloride, pickle liquor (containing substantial amounts of ferrous sulfate or ferrous chloride), and alum or hydrated aluminum sulfate.

The quantities of any of the various chemicals used for the removal of phosphorus increases rapidly as the required phosphorus reduction is increased. For example, to achieve a 75 per cent reduction in phosphorus, a weight ratio of alum to phosphorus is 13:1. To achieve a 95 per cent reduction, the weight ratio increases by 70 per cent to 22:1. As the dosage of alum or any other chemical used is increased, so is the cost of operation.

In summary, phosphorus control of eutrophication in C-SELM flowing streams only begins when removal has exceeded 90%. Furthermore, the cost for currently applied phosphorus removal technology at a treatment plant is controlled by the target level of residual phosphorus concentration rather than the amount of phosphorus removed. For example, it is just as costly to reduce phosphorus from 10 mg/l to 0.10 mg/l as it is to reduce phosphorus from 1 mg/l to 0.10 mg/l. Thus, when flowing streams receive significant portions of their flow from treated secondary effluent, as they do in much of the C-SELM area at present, the banning of phosphates in laundry detergents from the wastewater in these areas cannot be expected to retard algal blooms nor can it relieve subsequent anticipated treatment costs at the treatment plant.

A limited reduction of algal growth can be most easily accomplished in secondary effluents or in streams primarily consisting of secondary effluent by reductions in carbon, the limiting nutrient. This can be accomplished through advanced waste treatment procedures.

A reduction of algal populations to the background levels which can still be found in some relatively unpolluted lakes and streams as, for example, Lake Michigan open water, requires very low background

concentrations of phosphorus, approaching 0.01 mg/l. At these levels
of algal activity, only phosphorus can be used to control the algal
populations since carbon from natural bicarbonate sources and nitrogen
from fixation are present at too great a natural background level to
permit their use as controlling nutrients. Thus, a phosphate ban affect-
ing the quality of wastewater flows tributary to Lake Michigan could be
of value as an alternative to phosphorus removal at the treatment plant.
Return flows to Lake Michigan however should ultimately have much
higher degrees of phosphorus removal than are contemplated by those
procedures. Therefore, these procedures can only be regarded as interim
measures which should be supplanted as soon as practicable by more
extensive phosphorus removal technology.

The only practicable way at present to achieve background
phosphorus concentrations of 0.01 mg/l through water reclamation is
by use of the land process. Alternative advanced waste treatment and
physical-chemical methods yield residual phosphorus concentrations of
0.1-0.2 mg/l. The latter phosphorus concentration will support algal
populations ten to twenty times as large as can exist in unpolluted
background concentration water at 0.01 mg/l of phosphorus.

CONTROL OF PLEASURE AND COMMERCIAL WATERCRAFT WASTES

Bans on discharges from pleasure and commercial watercraft
contribute toward the reduction of pollutants in our waterways, whether
they be our major lakes and rivers, or smaller lakes and streams.
These discharges can be considered to be of two basic forms: human
and food wastes, and oily, bilge and ballast wastes.

Human and food wastes, in sufficient concentration in any
of our fresh water supplies, add unnecessarily to the already sizable
problem of filtration and purification to achieve a water quality safe
for human consumption. Quoting from a recent study, "Sanitary wastes,
including sewage, may carry pathogenic organisms which may cause
a variety of diseases such as dysentery, typhoid fever, and infectious
hepatitis. Concentrations of such wastes make water dangerous for
contact sports such as swimming and water skiing."[40]

With pleasure water craft becoming more and more popular and
abundant in our inland lakes and streams and along the shore of Lake
Michigan near the water intake cribs, it is only logical that they
should be restricted from discharging their wastes into the water supply
of millions. Also, wastes from the ocean and lake-going cargo ships

which ply Lake Michigan are likely to be a combination of human and food waste mixed with various bilge wastes. These bilge wastes are particularly objectionable since they may contain anything from bunker C fuel oil to refuse from any of a multitude of varieties of cargos and, may serve to transfer disease-bearing organisms.

In addition to making the production of potable water from lake water more difficult at the filtration plant, bilge wastes create serious problems for waterfowl and fish alike. These problems have been the subject of many documentaries, reports, and magazine and newspaper articles for several years.

Prevailing winds from the north cause surface films of oil and other debris, as well as ice floes in the winter, to drift or be pushed to the south end of Lake Michigan and worsen the situation.

Controlling pollution from the watercraft may be accomplished by means of either (1) on-board treatment of wastes to meet discharge standards, or (2) storing wastes on board to be later brought to shore by where they can be discharged into suitable facilities.

The installed cost of onboard equipment for properly handling watercraft wastes could vary from $40 to $100,000 per vessel, depending on size, type, mission, and other factors.

Space limitations on many craft prohibit the installation of adequate onboard treatment facilities for wastewater. This is especially true of small pleasure craft, where space is usually severely restricted and limited. For this reason, and due to the high cost of onboard facilities for small pleasure craft, onshore receiving facilities are a necessity, at least for the present.

To implement onshore discharge of onboard wastes, adequate and convenient facilities can be provided for the acceptance of these wastes. These facilities could be located on the inland waterways as well as on Lake Michigan, on the smaller lakes, and streams in the area large enough for pleasure craft. Wastes typically require sophisticated treatment processes which are able to cope with the many varieties of wastes likely to be discharged.

The Environmental Protection Agency (EPA) of the State of Illinois has adopted rules and regulations for the disposal of sewage from marine toilets. These rules and regulations have been reproduced as publication SWB-19.

SWB-19 states that acceptable pollution control devices are as follows:

1. Holding tanks which retain toilet wastes for proper disposal pursuant to Rule 3.3 (of SWB-19).

2. Incinerating devices which will reduce to ash all sewage and toilet wastes produced on the watercraft. Ash is to be disposed of on shore, not to waters of the State.

3. Any other device determined by the Illinois EPA to be effective in preventing pollution from marine toilets.

Waste control from commercial vessels is covered by this concise statement:

"Contaminated and pollutional bilge or ballast wastes shall be discharged only to appropriate shore facilities".

The State of Indiana has adopted similar measures for the control of discharges from pleasure and commercial watercraft.

The federal government has, as its basic control of discharges into waters under its jurisdiction, the Refuse Act of 1899 (Section 407).

In addition, dumping of wastes into Lake Michigan is controlled (and prohibited) by the Four State Conference. These conferees are the bordering states of Wisconsin, Illinois, Indiana, and Michigan.

Lake and ocean-going vessels operating on Lake Michigan are also under control of the International Joint Conference.

The State of Illinois EPA reports that the City of Chicago assists them in the enforcement of the provisions of SWB-19 with their requirement that watercraft moored at city harbors must comply with state statutes for the disposal of sewage from marine toilets before they will be allowed mooring space. For this reason, known violations of the provisions of SWB-19 are kept at a minimum.

Facilities are available at the various harbors in the Chicago area for the discharging of wastes from marine toilets, although they are admittedly too few for the number of watercraft in use. Facilities are also available along the Illinois waterway for receiving discharges from commercial vessels.

CONTROL OF WATER POLLUTION BY SOLID WASTE LANDFILLS

Introduction

The potential pollution of groundwaters from the decomposition products of buried refuse is a problem with dimensions not completely understood. There have been local examples of water quality impairment of a serious nature. This is true in spite of the fact that approximately 80 per cent of northeastern Illinois is suitable for sanitary landfilling with little site modification. Surficial materials are fine textured and have a low permeability, and would thus restrict the movement of leachate. Another ten per cent of the land area would be suitable because it is hydrologically well located. Sites in the remaining ten per cent of the land area would require a considerable amount of site modification and engineering to make them suitable for refuse landfills. Unfortunately, a large percentage of the sites proposed for sanitary landfills fall into this category. Mined out quarries and gravel pits dominate this latter group because of the financial gains that are possible. They are not safe for this purpose, however, unless site modification can be made.

The Effect of Leachate

The dissolved solids in the leachate from a landfill can travel with the groundwater and may, under certain circumstances, so degrade the groundwater that it can no longer be used for domestic purposes. Gases, predominantly methane and carbon dioxide, also are produced, with the latter capable of increasing the hardness of the groundwater. The problem remains long after the landfill operation has ended. Some landfills stabilize in a few years while others still produce methane and leachates after 30 years.

The amount of leaching depends on the type of refuse, the rate at which water contacts the refuse, and the refuse permeability, particle size, and relative compaction. A California study reports that wells 100 feet from a landfill were contaminated as well as areas more than one mile downstream.[41] Other studies document deterioration of surface waters 2.5 mi. distant. Although landfills can be theoretically free from this problem, it is not safe to assume so without checking. Unfortunately, regulatory and operating practices include inadequate or no monitoring and problems are not recognized until they have become severe.

Two water movements need to be controlled; precipitation which can percolate through refuse into the ground or surface water basins, and underground water which makes direct contact with buried materials. Under typical landfill conditions in Northeastern Illinois, about half the yearly precipitation will infiltrate the landfill surface. If this water, in the form of leachate, moves downward, groundwater mounds are formed. Such mounds have been found in disposal sites investigated by the Illinois State Geology Survey and reported in April, 1971. Groundwater mounds may result in the formation of springs around the margin of the filled area.

The Effectiveness of Control

Technology is available to handle all problems associated with solid waste disposal with relatively little expense and inconvenience. The major problem appears to be that of implementing this technology and regulating and supervising disposal operations.

Regulations and controls should:

1. Divert external surface runoff. Assure provision of an external control system to divert all surface runoff around the site. Creeks and drains into the area can be diverted around the refuse cell area.

2. Restrict precipitation from percolating through refuse. The precipitation which would normally be absorbed can be reduced through several methods.

   a. A nonpermeable cover material as advocated by national standards is not always adequate in itself. This type cover material is often not available.

   b. The planting of grasses with thick roots can absorb water, thus diverting it and minimizing the leachate formation. This is done at intermediate and final cover stages.

   c. Shredding of refuse increases the density of the material in the landfill and restricts percolation.

3. Provide for the interception and treatment of percolate.

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TECHNICAL APPENDIX B

COST ESTIMATION

V. COST ESTIMATION

A. METHODOLOGY

This section outlines the methods used to determine individual component costs which are presented in Appendix B, Section VI, and total, alternative management system costs presented in Appendix D.

BASE COSTS

Base costs do not include allowances for contingencies, engineering, or administrative fees. Base costs are developed for each of three major items; capital, operation and maintenance (O & M), and replacement costs. All base costs are indexed to an EPA Sewerage Treatment Plant Index of 180 for actual treatment plant costs, and to an ENR Construction Cost Index of 1,850 for all other. Figure B-V-A-1 presents a five-year plot of the respective indices. Actual values for each index are represented by closed points, while projected values are shown as open points.

Capital Costs For Construction

Capital costs are obtained from the application of unit capital cost figures. Unit capital costs reflect information from such sources as actual construction bids, estimates of research and pilot plant programs, and other published unit cost data. Specific sources of unit costs are referenced in the respective sub-sections of Appendix B, Section VI, which follows.

Capital Costs For Land

Capital costs associated with land include two components, one for actual purchase on a fee basis, the other for a fixed, initial payment for land that is not purchased but is utilized. For example, an initial payment equal to ten percent of the market value of the land is paid to the farmer whose land is used as a spray irrigation site. Another example is the fixed, lump-sum payment made to farmers where physical-chemical sludge is applied to their land. This lump-sum payment is a single payment equal to the market value of the farmer's land, in exchange for the long-term commitment for use of the land for the beneficial application of the soil conditioning, physical-chemical sludges.
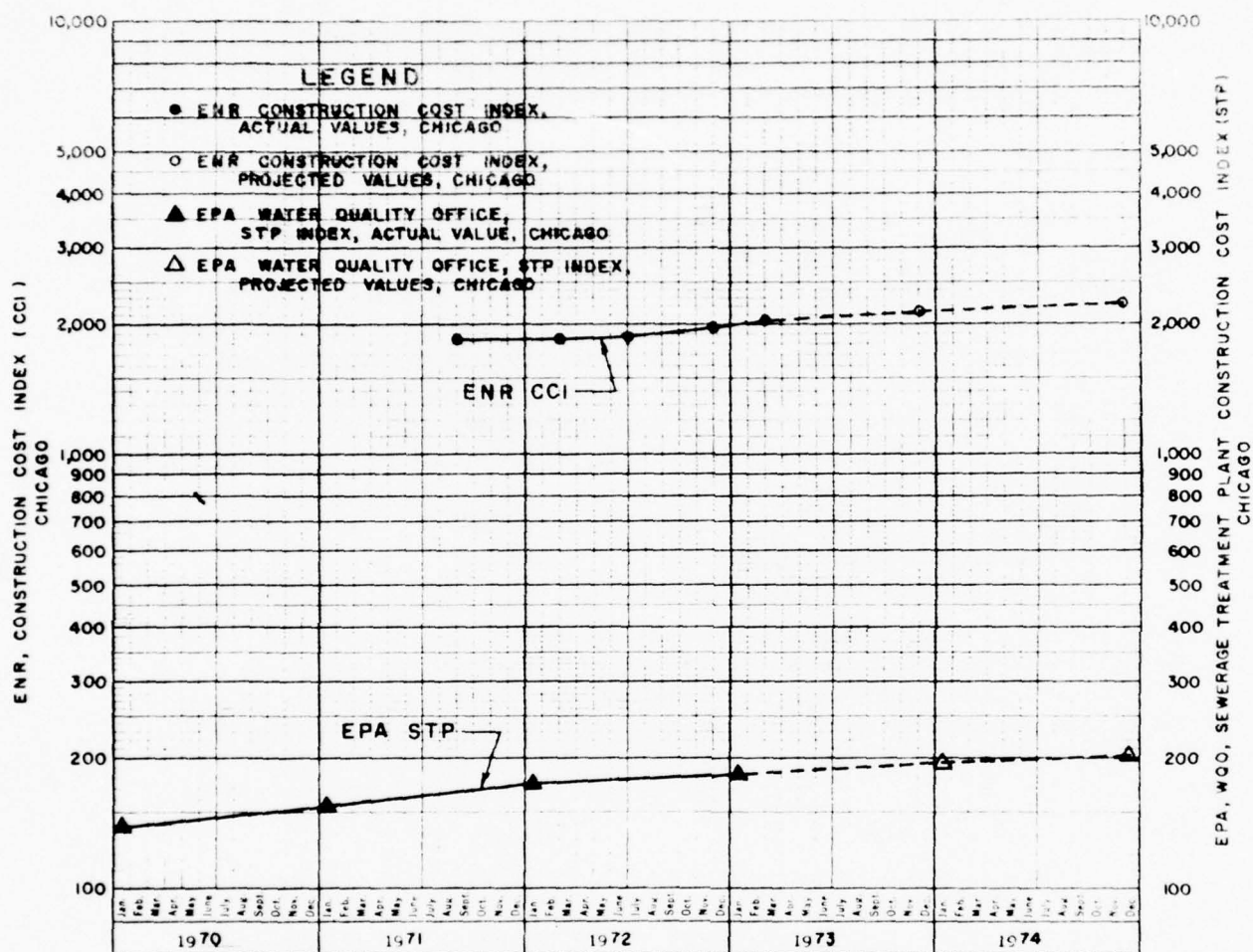
B-V-A-1

Figure B-Ⅴ-A-1
ENR CONSTRUCTION COST INDEX, CHICAGO
EPA, WQO SEWERAGE TREATMENT PLANT COST INDEX, CHICAGO

B-V-A-2

Operation And Maintenance Costs (Land Not Included)

O & M costs reflect actual operating data from pertinent, or analogous units, or estimates from research and pilot programs. Specific unit O & M costs are referenced in Appendix B, Section VI. A brief study of comparative operation and maintenance costs is presented in Data Annex BA, Section V.

Operation And Maintenance Costs (Land)

An annual fixed payment, which can be characterized as an O & M cost, is included for all land that is utilized in a particular system alternative, but not purchased in fee for the system. For example, when a farmer's land is being used as a spray irrigation site, he is paid an annual use fee equal to four percent of the market value of his land. The market value used in this analysis is the value at the time of original entrance into contract with the system. This payment is to compensate the owner for the land appreciation that he could otherwise realize by sale of the land.

Replacement Costs

Replacement costs reflect schedules which have been established on an individual basis for major system inputs to the overall management system design.

These individual replacement schedules are presented in Appendix B, Section VI.

CONTINGENCIES, AND ENGINEERING AND ADMINISTRATIVE COSTS

General

Contingencies, and engineering-and-administrative (E-&-A) costs are added to base costs. Values used are 20 percent for contingency costs and 15 percent for E-&-A fees. The application of these allowances is presented below.

Capital Costs For Construction

Base capital costs plus contingencies plus E-&-A, i.e., base costs + 20% + 15%, form total capital costs exclusive of land capital costs.

Capital Costs For Land

Purchased land capital costs include land base capital costs plus contingencies plus E-&-A, i.e., base cost + 20% + 15%. Utilized land costs include only contingency cost.

Operation And Maintenance Costs (Land Not Included)

O & M costs include base O & M costs plus the 20% contingency allowance.

Operation And Maintenance Costs (Land)

The annual fixed payment to owners of utilized land includes an added 20% contingency allowance.

Replacement Costs

Replacement costs determined from base capital costs by individual replacement schedules include an added contingency allowance of 20%.

ECONOMIC ANALYSIS CONSIDERATION

The economic life of the system is fifty years, with implementation assumed to begin on January 1, 1975. Four interest rates are used in Appendix D. These are 5, 5-1/2, 7 and 10 percent. Two analyses are performed on the Appendix D total cost data, a present worth analysis and a total average annual cost analysis.

PRESENT WORTH

The present worth analysis is based upon standard present worth evaluation techniques. Total capital costs are assumed to be expended over a planned construction period, which is outlined in Appendix D. Operation-and-maintenance (O & M) costs are returned to the zero year of 1975 by a gradient analysis which reflects annually increasing O & M costs associated with increased flows. Replacement costs are returned to the zero year from their year of implementation. Total present worth equals the sum of these three costs.

ANNUAL COST

The annual cost analysis is also based upon standard calculation methods. Total capital costs are amortized over the 50-year

economic life of the system. Annual O & M costs which vary with time are converted to a uniform annual series of costs. Replacement costs are brought back to the zero year through a present worth analysis and then converted to a uniform annual series of expenditures. Total average annual costs reflect the sum of these three costs.

TECHNICAL APPENDIX B

COMPONENT BASIS OF COST

VI. COMPONENT BASIS OF COST

A. REGIONAL TREATMENT SYSTEMS

INTRODUCTION

Presented in this section are the detailed cost bases which are utilized for cost evaluations of the modular advanced waste treatment systems. The three AWT systems are evaluated on a unit cost basis. The unit costs include an initial capital or construction cost together with a present-worthed replacement cost in dollars/MGD of treatment capacity. An operation and maintenance unit cost is also presented in dollars/MG of wastewater treated. The detailed costs of the unit processes are presented in this section followed by the summary costs for the treatment plant and land treatment systems.

The basis for the unit costs utilized in this study were largely developed in the Cost Data Annex of the OCE C-SELM model study. [1]/ Included in that annex were detailed unit costs, cost analyses and references.

For the unit treatment plant processes, actual available bid data *were used, as well as the cost estimates* that have been generated through research and pilot plant operations. Construction unit cost estimates for other items were based on available bid data. For the land treatment alternatives, extensive use of costs for the Muskegon County, Michigan Wastewater Management System is made.

Land acquisition costs for the land treatment system are presented in detail in Appendix B, Section VII-A under the subsection of land displacements.

All costs have been adjusted to an EPA STP Index of 180 or an ENR Index of 1850.

Estimates of operation and maintenance costs were based on available actual cost data and on estimates from advanced treatment process research and pilot plant operations. Replacement cost estimates were based on the information developed in the OCE C-SELM model study.

TREATMENT PLANT SYSTEMS - CAPITAL COSTS

Introduction

The following unit process costs relate to the 100-MGD modular designed AWT plant. Thus all per MGD or per MG unit costs are for a 100 MGD capacity plant. Based on recirculation factors, the unit process capacities may be greater than 100 MGD. However, these recirculation flows are taken into account in the component AWT cost graphs. Unit costs for AWT components less than 100 MGD, as shown in the cost graphs, reflect diseconomies in scale. The costs presented in this section do not reflect sludge disposal costs, contingencies, engineering design, legal and administration fees and land costs. For a basis of comparison with the land treatment system, land costs, landscaping, interconnection piping, administration and lab buildings are assumed to equal 10% of the capital cost of the treatment facilities.

Secondary Treatment

As mentioned in the component basis of design, conventional secondary treatment (activated sludge) includes pretreatment facilities and lift stations, primary and secondary treatment facilities, sludge digestion and storage facilities and, finally, chlorination units. For a 100 MGD capacity plant, the detailed capital costs are:

Mechanical Bar Screens, Lift Station & Pumping Facilities	$ 1.0 Million
Aerated Grit Tanks and Grit Removal Facilities	0.6 "
Primary Clarification Tanks & Sludge Collection Equipment	4.6 "
Aeration Tanks, Diffuser Piping, & Blower Building	17.4 "
Final Clarification Tanks, Return & Waste Sludge Pumps	8.3 "
Chlorination Facilities	0.5 "
Sludge Digestion & Storage Facilities	6.6 "

Total Secondary Treatment Capital Cost $39.0 Million

Unit Process Capital Cost = $390,000/MGD

Nitrification-Denitrification

The capital costs for this unit process include concrete structural tanks and related air diffusers, chemical feed and sludge pumping equipment. The capital costs for the 100 MGD module are:

Nitrification Reactor Basins, Diffuser Piping & Blower Building	$ 8.4 Million
Nitrification Settling Tanks & Return Sludge Pumps	10.8 "
Denitrification Reactor Basins, Mixers & Methanol Feed Equipment	2.0 "
Denitrification Settling Tanks, Return & Waste Sludge Pumps	10.8 "
Total Nitrification-Denitrification Capital Cost	$32.0 Million

Unit Process Capital Cost = $320,000/MGD

Lime Clarification

The capital costs for this unit process include lime feed and storage facilities, rapid mix basins, lime reactor clarifiers, recarbonation reactor clarifiers, sludge pumping facilities, sludge thickening and dewatering facilities, lime recalcination units, carbon dioxide compressors and a solids (ash) classification and handling system. The above-mentioned facilities are designed for 140 MGD since all recirculated flows from this modular design enter the lime process. The capital cost for this 140-MGD lime clarification system is presented below:

Lime Feed & Storage System	$ 0.5 Million
Lime Rapid Mix, Flocculation, Clarification and Sludge Pumps	3.8 "
Recarbonation Reactor-Clarifiers, Carbon Dioxide Compressors, Diffuser Piping & Sludge Pumps	4.0 "
Sludge Thickening, Dewatering, & Conveyance Facilities	4.9 "
Lime Recalcination Furnace & Solids Classification System	5.0
Total Lime Clarification Capital Cost	$18.2 Million

Unit Process Capital Cost = $182,000/MGD

Carbon Adsorption System

The capital costs for the carbon adsorption system include the structural cost of the carbon contactors, the pumps, piping and instrumentation facilities, the carbon regeneration facilities and the cost of the carbon inventory. The costs presented below reflect a total process flow, including recirculation, of 130 MGD for the modular design. Since the organic loading is different between the two treatment plant systems, the regeneration costs for these systems are not identical.

Carbon Contact Columns & Building	$ 7.9 Million
Wastewater Pumps, Piping and	6.1 " (P-C)[a]
Instrumentation	5.5 " (A-B)[b]
Multiple Hearth Regeneration Furnace	2.7 " (P-C)[a]
Wet Scrubbers, Quench Tanks &	
Carbon Slurry Pumps	2.2 " (A-B)[b]
Granular Carbon Inventory	6.0 "

Total Carbon Adsorption Capital Cost
(Physical-Chemical) $22.7 Million

Total Carbon Adsorption Capital Cost
(Advanced Biological) $21.6 Million

Unit Process Capital Cost
(Physical-Chemical) = $227,000/MGD

Unit Process Capital Cost
(Advanced Biological) = $216,000/MGD

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[a]From hereout P-C will refer to Physical-Chemical Treatment System.

[b]From hereout A-B will refer to Advanced Biological Treatment System.

Clinoptilolite Ion Exchange

The clinoptilolite ion exchange process is designed to remove ammonia from the wastewater. This unit process includes clinoptilolite steel exchange columns, chemical feed and storage facilities, regenerant pumps and piping facilities, air stripping towers and the clinoptilolite inventory. The capital costs presented below are for a unit process capacity of 110 MGD.

Clinoptilolite Exchange Columns, Building
       & Clinoptilolite Inventory $ 8.2 Million

Chemical Feed, Storage, Mixing Basin
       & Regenerant Storage Tanks   1.0   "

Elutrient Pumps, Valves and Instru-
       mentation   4.5   "

Air Stripping Tower, Heat Exchanger &
       Boiler   4.5
                                            ———————

Total Clinoptilolite Ion Exchange
       Capital Cost $18.2 Million

Unit Process Capital Cost = $182,000/MGD

Mixed-Media Filtration

This filtration process includes capital cost items such as concrete tank and building structures, filter material, backwash pumps, underdrains and related piping. The costs presented below are for a filter capacity of 110 MGD.

Filter Tanks & Building Structure $ 5.0 Million
Underdrain and Backwash System   0.6   "
Filter Media Material   0.3   "
                                            ———————

Total Mixed Media Filtration Cost $ 5.9 Million

Unit Process Capital Cost = $59,000/MGD

Chlorination

The chlorination facilities include the building structure, chlorine feed and storage facilities, chlorinators and evaporators. For the modular treatment plant design, the capacity of this unit process is 100 MGD.

Chlorine Contact Tanks & Building $ 0.45 Million
Storage & Chemical Feed System   0.05   "
                                            ———————

Total Chlorination Capital Cost $ 0.50 Million

Unit Process Capital Cost = $5,000/MGD

Post Aeration

The capital costs for this unit process include concrete tanks, mechanical surface aerators and related electrical facilities. The capacity of this design module is 100 MGD with the following capital costs:

Concrete Tank Structure	$ 1.9 Million
Mechanical Surface Aerators	0.7 "
Total Post Aeration Capital Cost	$ 2.6 Million

Unit Process Capital Cost = $26,000/MGD

Pretreatment

This unit process, which is included in the secondary component of the advanced biological system, is also a treatment component of the physical-chemical treatment system. Included in the capital cost of this process are mechanical bar screens, lift station, aerated grit tanks and grit removal facilities.

Mechanical Bar Screens, Lift Station Facilities	$ 1.0 Million
Aerated Grit Tanks & Grit Removal Facilities	0.6 "
Total Pretreatment Capital Cost	$ 1.6 Million

Unit Process Capital Cost = $16,000/MGD

TREATMENT PLANT SYSTEMS - REPLACEMENT COSTS

Introduction

Replacement costs refer to programmed capital expenditures for certain treatment components whose design life is less than 50 years, which is the life of our treatment plant system. The following replacement costs for the various unit processes relate to the aforementioned capital costs based on a modular 100-MGD plant capacity design. Replacement costs are presented in this section as a percentage of the capital costs of the particular treatment component. The programmed replacement schedule is also presented in this section.

Secondary Treatment

Chlorination feed systems, lift and sludge pumps, grit convey-
ors and blowers are replaced every ten years in the secondary treatment
facilities. This approximates a 7.5% capital expenditure of the total
secondary treatment facility for four times during the life of the system.
Also included in the secondary treatment replacement cost is a major
overhaul of the plant after 25 years. This includes suuctural repair of
concrete tanks, digesters and related piping systems at a cost equal to
25% of the total facility capital cost.

Nitrification-Denitrification

Similar to the previously mentioned secondary facilities, the
blowers, sludge pumps, mixers and chemical feed systems are replaced
every ten years at a cost equal to 10% of the total capital cost of the
nitrification-denitrification facilities. After 25 years, a replacement
and major repair of the concrete tanks and related piping is programmed
at a cost equal to 20% of the total facility cost.

Lime Clarification

Every five years, or nine times during the life of the system, replace-
ment and major repairs to the lime recalcination furnace and carbon dio-
xide compressors and lime sludge pumps will take place. The cost of
this five-year replacement is equal to 10% of the total capital cost of
lime clarification facilities. Replacement of lime feed and rapid mix
components, sludge dewatering facilities and lime clarification facilities
(structural repair and sludge collection replacement) occurs every ten
years at a cost equal to 15% of the total capital cost.

Carbon Adsorption

Replacement of various components of the multiple hearth regen-
eration facilities at a cost equal to 5% of the total, is programmed to
take place every five years for the life of the system. Replacement of
the wastewater and carbon slurry pumps and related piping takes place
every ten years at a cost equal to 7.5% of the total facility cost. Once
during the life of the system, after 25 years of operation, major struc-
tural repair to the steel carbon adsorption columns, defining and quench
tanks occurs at a cost equal to 10% of the total capital cost.

Clinoptilolite Ion Exchange

Replacement of capital facilities of this unit process is similar to that of the carbon adsorption system. Every five years, 5% of the total facility cost is replaced. This is for the ammonia air stripping tower and related facilities such as heat exchangers and boilers, etc. Replacement of regenerant pumps and chemical feed and mixing processes occurs every ten years at a cost equal to 7.5% of the total. Finally, once in 25 years, replacement and major repair of the ion exchange columns, mixing and recycle basins and regenerant storage tanks take place at a cost equal to 10% of the total ion exchange unit process cost.

Mixed Media Filtration

Replacement of the mixed-media filter material and backwash pumps are programmed at ten-year intervals at a cost equal to 10% of the total system cost. Major structural repair to the concrete filter tanks is designed to take place after 25 years at a cost equal to 20% of the total unit process cost.

Chlorination

Replacement of chlorinators, evaporators, chemical feed systems and repair of storage facilities for the chlorination unit process takes place every ten years. This replacement cost is equal to 25% of the total unit process capital cost.

Pretreatment

This unit process replacement cost is included in the secondary treatment component of the advanced biological system. For the physical-chemical system, a ten-year replacement schedule for lift station pumps, screening and grit collection systems is programmed. This replacement cost is equal to 20% of the total unit process facility cost.

TREATMENT PLANT SYSTEMS - OPERATION AND MAINTENANCE COSTS

Introduction

The operation and maintenance costs of the treatment facility components include labor, chemicals and supplies and energy requirements. Due to the advanced technology and size of the modular treatment systems, round-the-clock maintenance of the facility must be

maintained. Taking into account vacation, sick time and weekends, 4.5 treatment plant shifts are necessary to insure proper performance on a year—round basis. The unit labor costs, which include insurance and FICA tax overhead rates, are broken down into three main categories:

(1)   Supervisors @ $20,000/year
(2)   Skilled Labor @ $15,000/year
(3)   Unskilled Labor @ $10,000/year

Included in the skilled labor category would be treatment plant operators, chemists, electricians and mechanics. Unskilled labor personnel includes general maintenance, laborers, lab technicians and clerk-typists. The following is a compilation of unit process O & M costs.

General Plant Functions

For the advanced biological and physical-chemical systems, there are general plant supervisory personnel whose function is to oversee the operation of the treatment facility. Such positions as administrator, superintendent, chief engineer and their assistants are included in general plant functions. Unskilled labor includes clerk-typists and maintenance personnel for the building and grounds. For the advanced biological system, the general plant labor requirements are as follows:

```
3 Supervisors              $ 60,000/year
6 Unskilled Labor          $ 60,000/year
    Labor Cost/shift       $120,000/year
    @4.5 shifts =          $540,000/year
Unit Labor Cost =  $15/MG
```

This labor cost is distributed to the various advanced biological treatment components based on the ratio of the unit process size to the whole system. The general plant functions unit labor cost distributions are $5/MG for secondary and chlorination facilities, $4/MG for nitrification-denitrification, $2/MG for lime clarification, $3/MG for carbon adsorption, $0.75/MG for the mixed-media filtration process and $0.25/MG for the post-aeration facilities.

For the physical-chemical system, the general plant labor requirements are:

```
       3 Supervisors              $ 60,000/year
       4 Unskilled Labor          $ 40,000/year
            Labor Cost/shift      $100,000/year
            @4.5 shifts =         $450,000/year
       Unit Labor Cost = $13/MG
```

This labor cost is likewise distributed to the physical-chemical treatment components as follows: $0.50/MG for pretreatment, $3.50/MG for lime clarification, $4/MG for carbon adsorption, $3.50/MG for clinoptilolite ion exchange, $1/MG for mixed media filtration, and $0.50/MG for post-aeration.

These unit labor costs are included in the following unit process O & M costs presented below:

Secondary Treatment

```
       Labor
       1 Supervisor              $ 20,000/year
      10 Skilled Labor           $150,000/year
       9 Unskilled Labor         $ 90,000/year
            Labor Cost/shift      $260,000/year
            @4.5 shifts          $1,170,000/year
       Unit Labor Cost
         (including General Plant
         Functions) = $37/MG
```

```
       Chemical & Supplies
       Chlorine @ 4 mg/l and $0.05/pound           $  1.75/MG
       Supplies (Lab equipment parts and
              misc. utilities) @1.0% Capital
              Cost/Year =                           $10.25/MG
                                                    ──────────
       Total Chemical & Supply Cost                 $12.00/MG
```

```
       Energy
       Lift Station @1,000 HP & $0.01/KWH           $  1.75/MG
       Aeration @3,100 HP =                            5.50/MG
       Sludge Pumping @500 HP =                        0.75/MG
                                                    ──────────
       Total Energy Cost                            $  8.00/MG
```

Total Secondary Treatment O & M Cost = $57/MG

Presented in Figure B-VI-A-1 is the O & M and capital cost curve for secondary treatment which was formulated based on the above detailed modular maximum economy of scale costs.

Nitrification-Denitrification

    Labor
    1 Supervisor $ 20,000/year
    3 Skilled Labor $ 45,000/year
    2 Unskilled Labor $ 20,000/year
          Labor Cost/shift $ 85,000/year
          @4.5 shifts = $383,000/year
    Unit Labor Cost = $15/MG

    Chemicals & Supplies
    Methanol @40 mg/1 $0.025/pound $ 9/MG
    Supplies @0.5% Capital Cost/Year   4/MG

    Total Chemical & Supply Cost $13/MG

    Energy
    Aeration @$3,500 HP & $0.01/KWH $ 6/MG
    Sludge Pumping & Denitrification Mixing
          @1,100 HP   2/MG

    Total Energy Cost $ 8/MG

Total Nitrification-Denitrification O & M
        Cost = $36/MG

The O & M and capital cost curve which relates to this unit process modular design is presented in Figure B-VI-A-2 together with unit costs for plant capacities as low as one MGD.

Lime Clarification

    Labor
    1 Supervisor $ 20,000/year
    5 Skilled Labor $ 75,000/year
    2 Unskilled Labor $ 20,000/year
          Labor Cost/shift $115,000/year
          @4.5 shifts = $518,000/year
Unit Labor Cost = $16/MG for the advanced biological system.
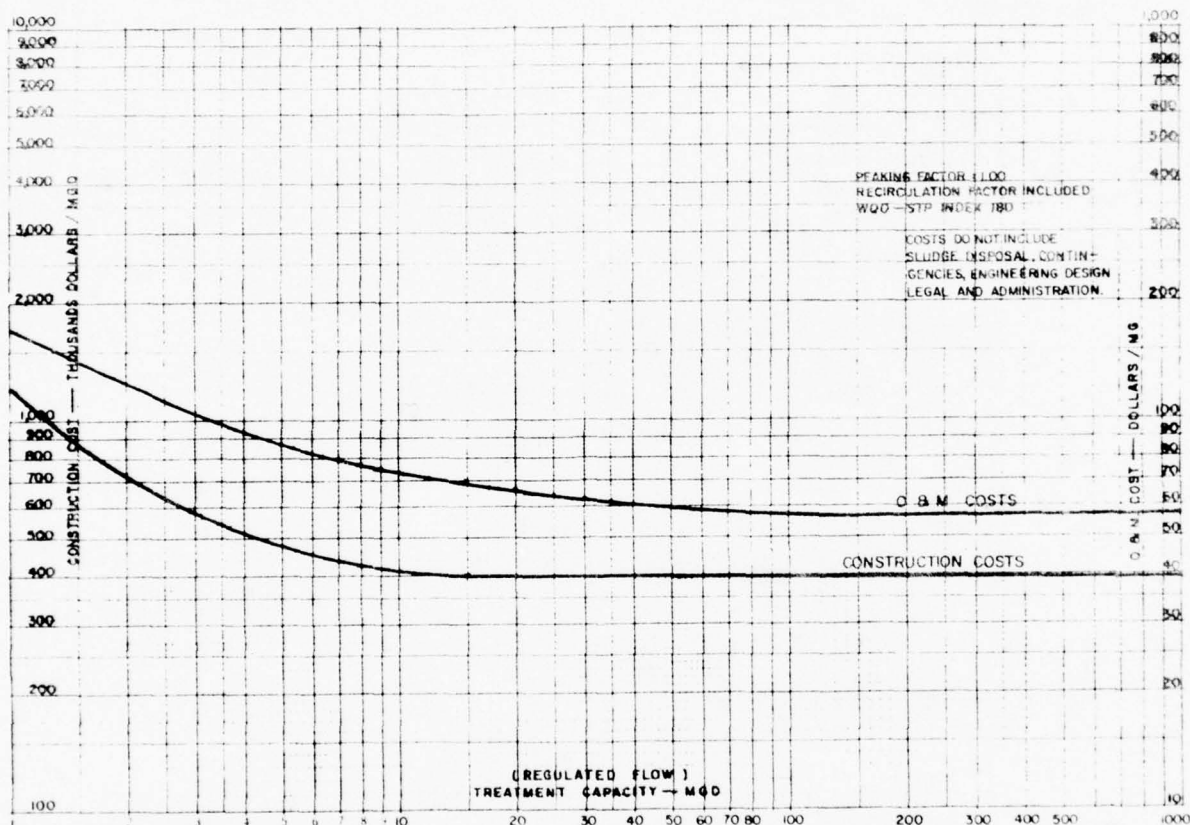
Figure B-VI-A-1

SECONDARY TREATMENT

COST CURVE
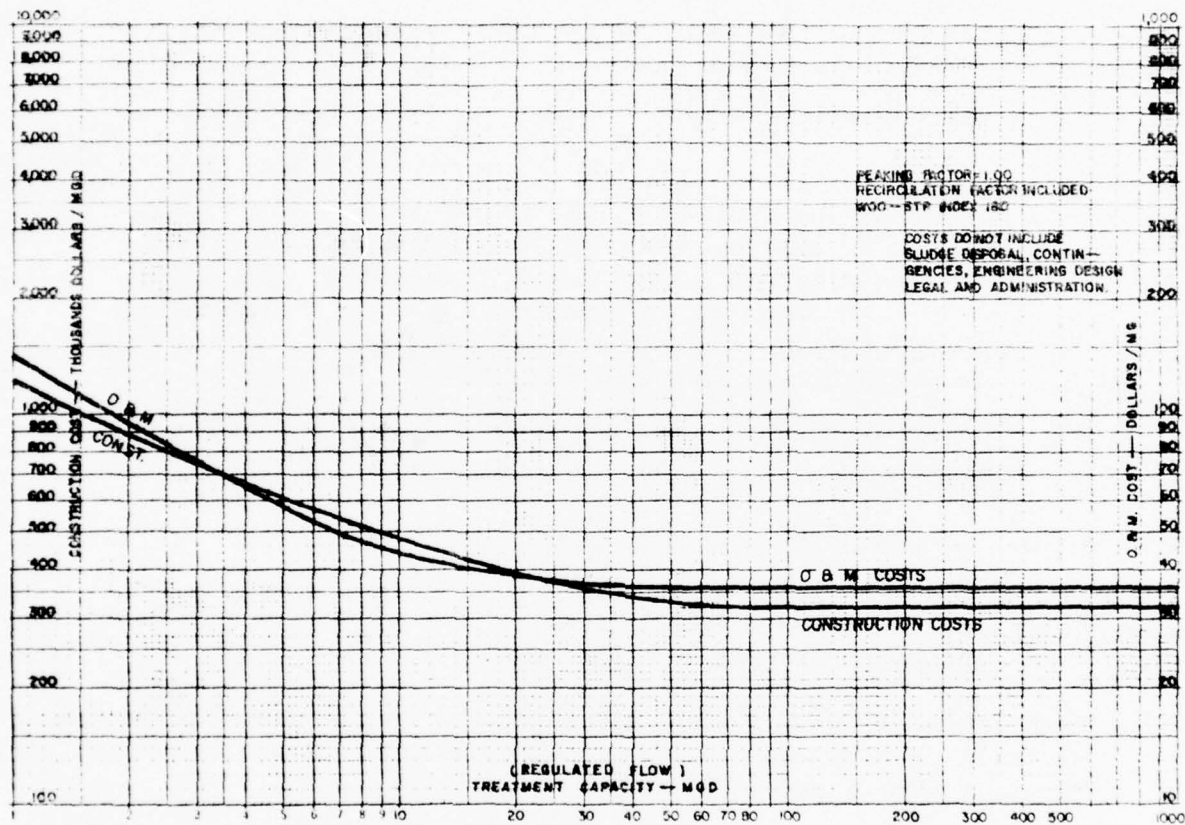
Figure B-VI-A-2

NITRIFICATION-DENITRIFICATION
COST CURVE

For the physical-chemical system, the unit labor cost increases to $17/MG. This is accounted for by the greater solids loading to this system and greater proportion of general plant function costs to this unit process.

<div style="margin-left: 2em;">

Chemicals & Supplies
Lime makeup @112 mg/l & 90% purity & $20/T	$15/MG
Supplies @0.67% of Capital Cost/Year	3/MG
Total Chemical & Supply Cost	$18/MG

Energy
Recalcination @32,000 ft^3 natural gas/MG for physical-chemical $0.50/1000 ft^3	$16/MG
Recalcination @26,000 ft^3 natural gas/MG for advanced biological	13/MG
Sludge Pumping & Dewatering @1,800 HP	3/MG
Recarbonation Compressors @600 HP	1/MG
Total Energy Cost (Physical-Chemical)	$20/MG
Total Energy Cost (Advanced Biological)	$17/MG

</div>

Total Lime Clarification O & M Cost
(Physical-Chemical) = $55/MG

Total Lime Clarification O & M Cost
(Advanced Biological) = $51/MG

These modular 100-MGD O & M and capital unit process costs for both treatment plant systems are graphically presented in the cost curves of Figures B-VI-A-3 and B-VI-A-4.

Carbon Adsorption

<div style="margin-left: 2em;">

Labor
1 Supervisor	$ 20,000/year
13 Skilled Labor	195,000/year
5 Unskilled Labor	50,000/year
Labor Cost/shift	$265,000/year
@4.5 Shifts =	$1,193,000/year
Unit Labor Cost =	$36/MG

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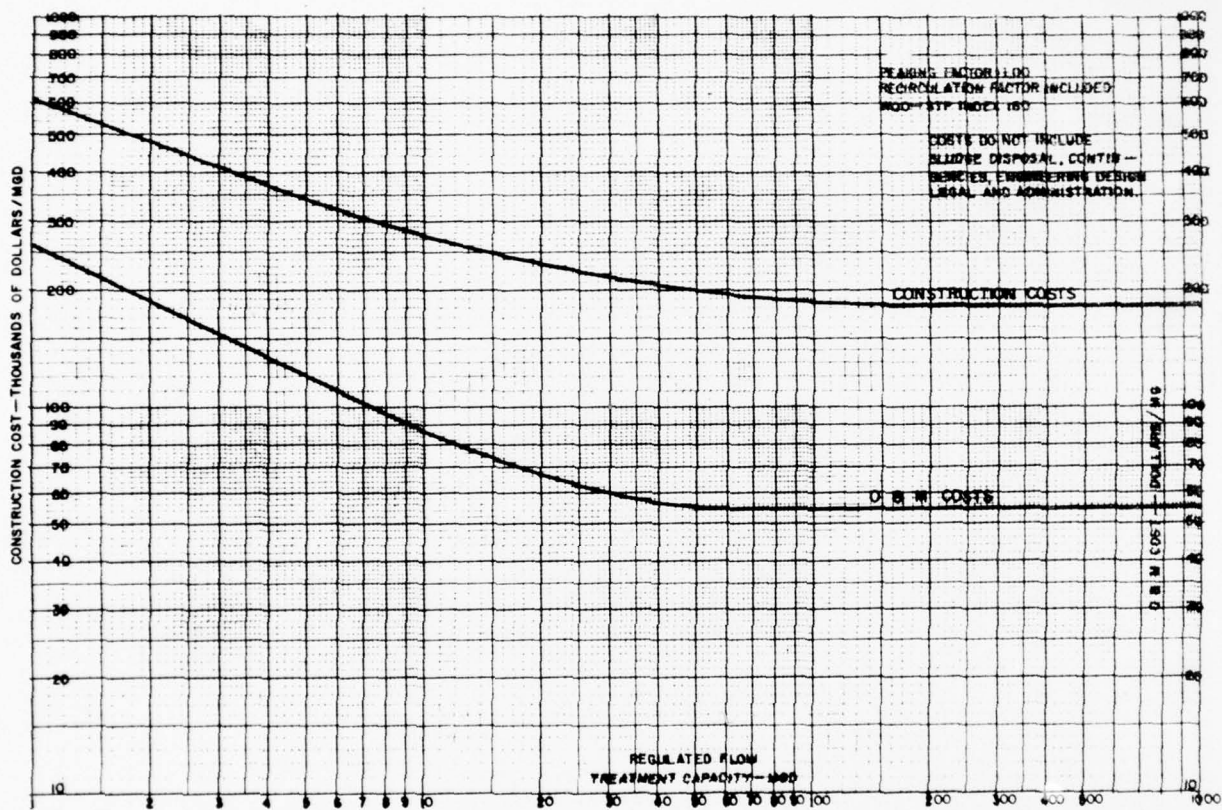
Figure B-VI-A-3

LIME CLARIFICATION--PHYSICAL-CHEMICAL
COST CURVE
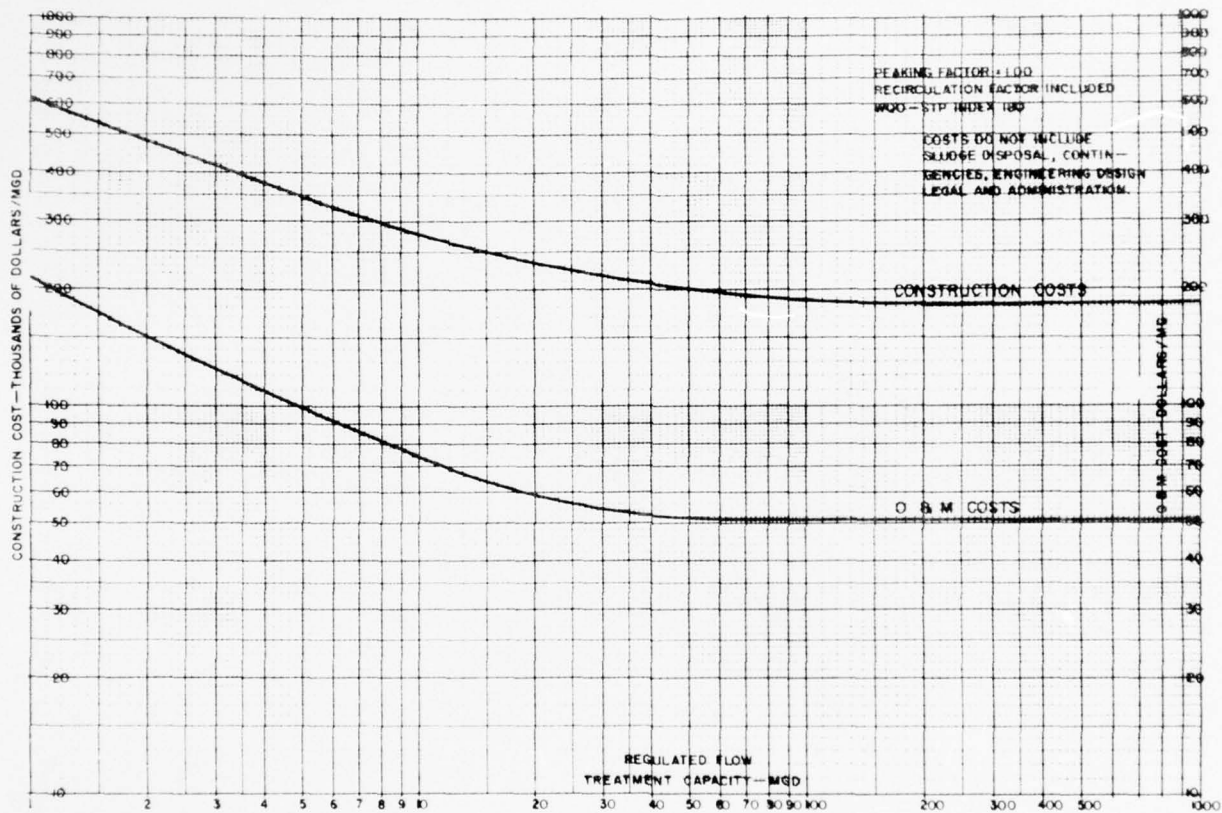
Figure B-VI-A-4

LIME CLARIFICATION--ADVANCED BIOLOGICAL
COST CURVE

<u>Chemicals & Supplies</u>

Carbon Makeup @68 pounds/MG @$0.28/MG for physical-chemical	$19/MG
Carbon Makeup @34 pounds/MG for advanced biological	10/MG
Supplies @0.67% Capital Cost/Year =	4/MG
Total Chemical & Supply Cost (Physical-Chemical)	$23/MG
Total Chemical & Supply Cost (Advanced Biological)	$14/MG

<u>Energy</u>

Regeneration @4,000 ft^3 natural gas/MG & $0.50/1,000 ft^3 for physical-chemical	$ 2/MG
Regeneration @2,000 ft^3 natural gas/MG for advanced biological	1/MG
Adsorption & Regeneration Wastewater Pumping @3,400 HP & $0.01/KWH	6/MG
Total Energy Cost (Physical-Chemical)	$ 8/MG
Total Energy Cost (Advanced Biological)	$ 7/MG

Total Carbon Adsorption O & M Cost
(Physical-Chemical) = $67/MG

Total Carbon Adsorption O & M Cost
(Advanced Biological) = $57/MG

Presented in Figures B-VI-A-5 and B-VI-A-6 are the above-mentioned modular O & M and capital costs of the carbon adsorption process for both treatment plant systems.

<u>Clinoptilolite Ion Exchange</u>

<u>Labor</u>

1 Supervisor	$ 20,000/year
8 Skilled Labor	$120,000/year
2 Unskilled Labor	$ 20,000/year
Labor Cost/shift	$160,000/year
@4.5 Shifts =	$720,000/year
Unit Labor Cost = $23/MG	
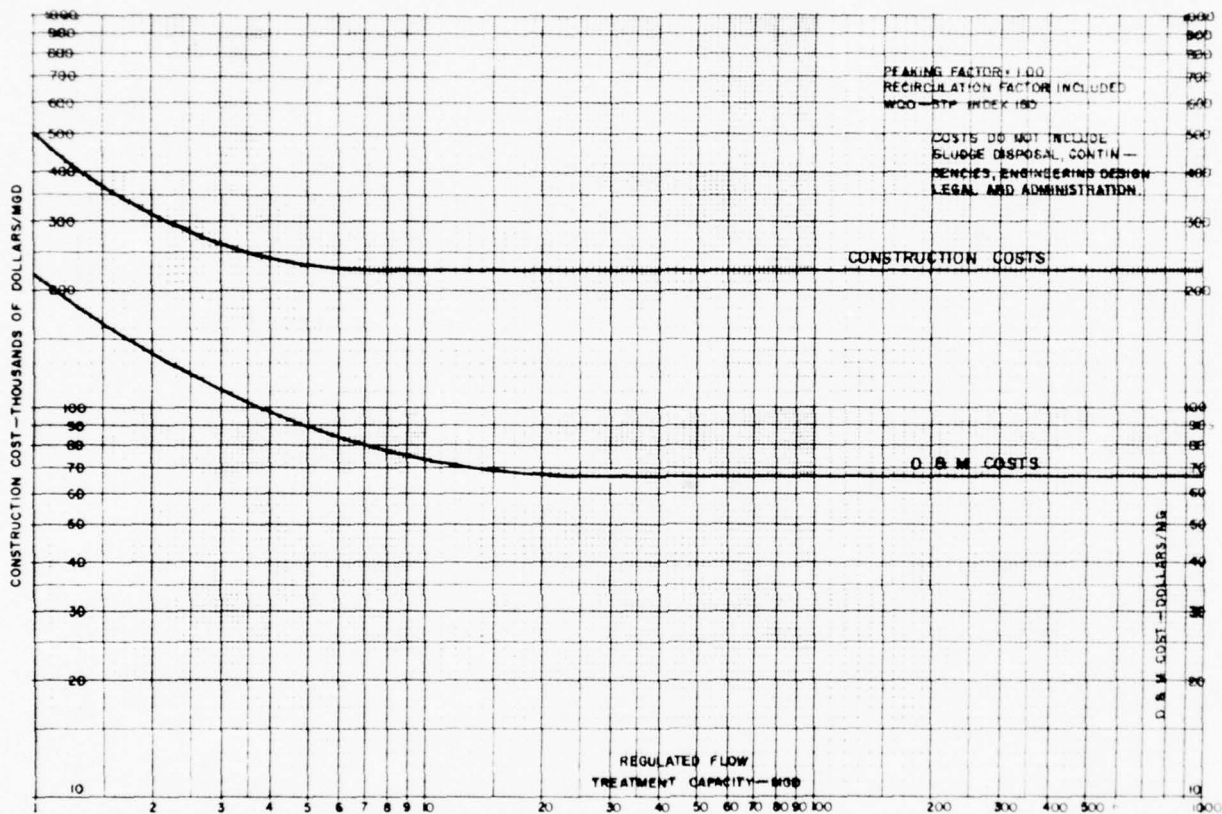
Figure B-VI-A-5

CARBON ADSORPTION & REGENERATION--PHYSICAL-CHEMICAL
COST CURVE

B-VI-A-18
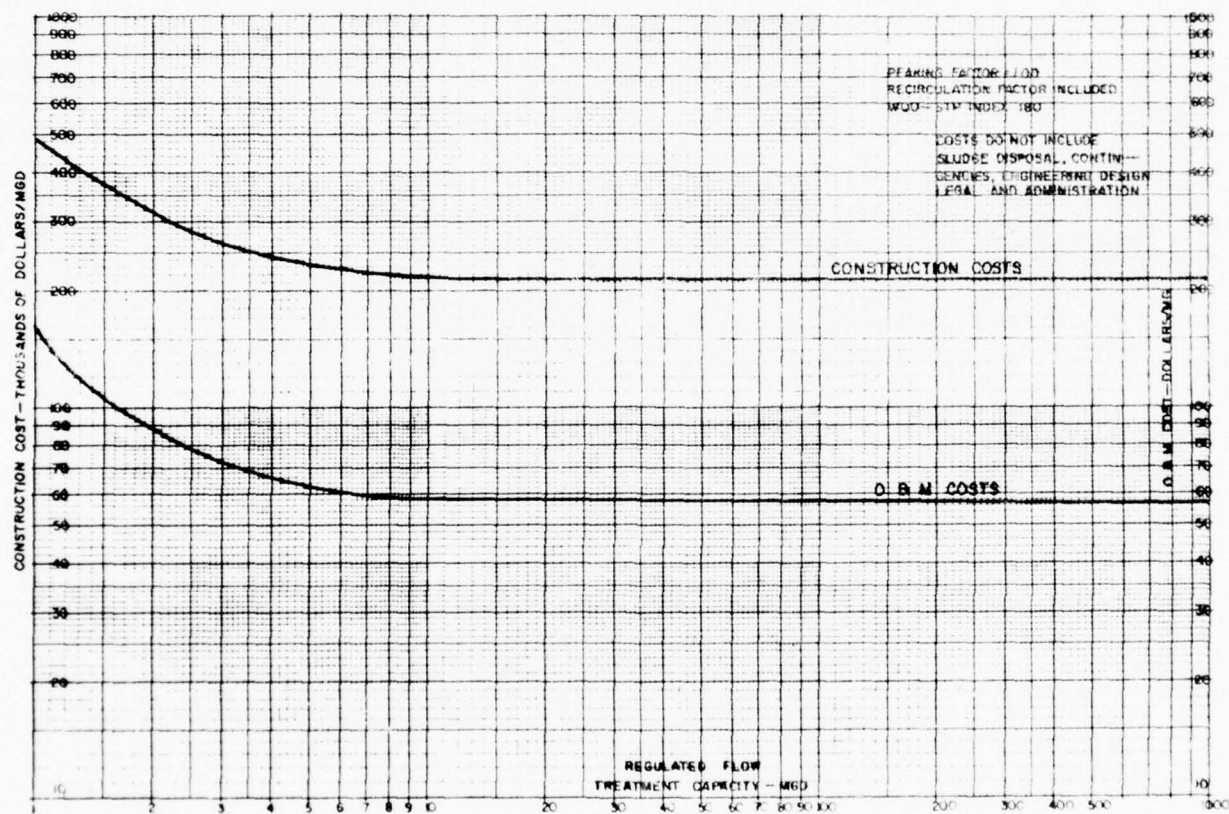
Figure B-VI-A-6

CARBON ADSORPTION & REGENERATION--ADVANCED BIOLOGICAL
COST CURVE

B-VI-A-19

Clinoptilolite Makeup @ 160 pounds/MG &	
$0.075/pound	$12/MG
Lime @50 mg/l & $20/ton	5/MG
Sodium Chloride @70 mg/l & $20/ton	6/MG
Supplies @1% Capital Cost	5/MG
Total Chemical & Supply Cost	$28/MG

Energy	
Air Blowers @6,000 HP $0.01/KWH	$10/MG
Gas Boiler @16,000 ft^3 natural gas/MG & $0.50/1,000 ft^3	8/MG
Total Energy Cost	$18/MG

Total Clinoptilolite Ion Exchange O & M Cost =	$69/MG

This modular 100-MGD O & M and capital cost for the ion exchange process is graphically presented in the cost curve Figure B-VI-A-7.

Mixed-Media Filtration

Labor

2.5 Skilled Labor	$ 37,000/year
1 Unskilled Labor	10,000/year
Labor Cost/shift	$ 47,000/year
@4.5 Shifts =	$212,000/year

Unit Labor Cost = $6.50/MG
          for the advanced biological system.

This labor cost increases to $6.75/MG for the physical-chemical system since this unit process has a greater share of the general plant function labor cost.

Chemicals & Supplies

Alum @18 mg/l liquid alum & $0.018/pound	$ 3/MG
Polymer @0.1 mg/l & $1.75/pound	1/MG
Supplies @0.67% of Capital Cost/Year	1/MG
Total Chemical & Supply Cost	$ 5/MG

Energy

Backwash pumping @100 HP & $0.01/KWH	$0.25/MG
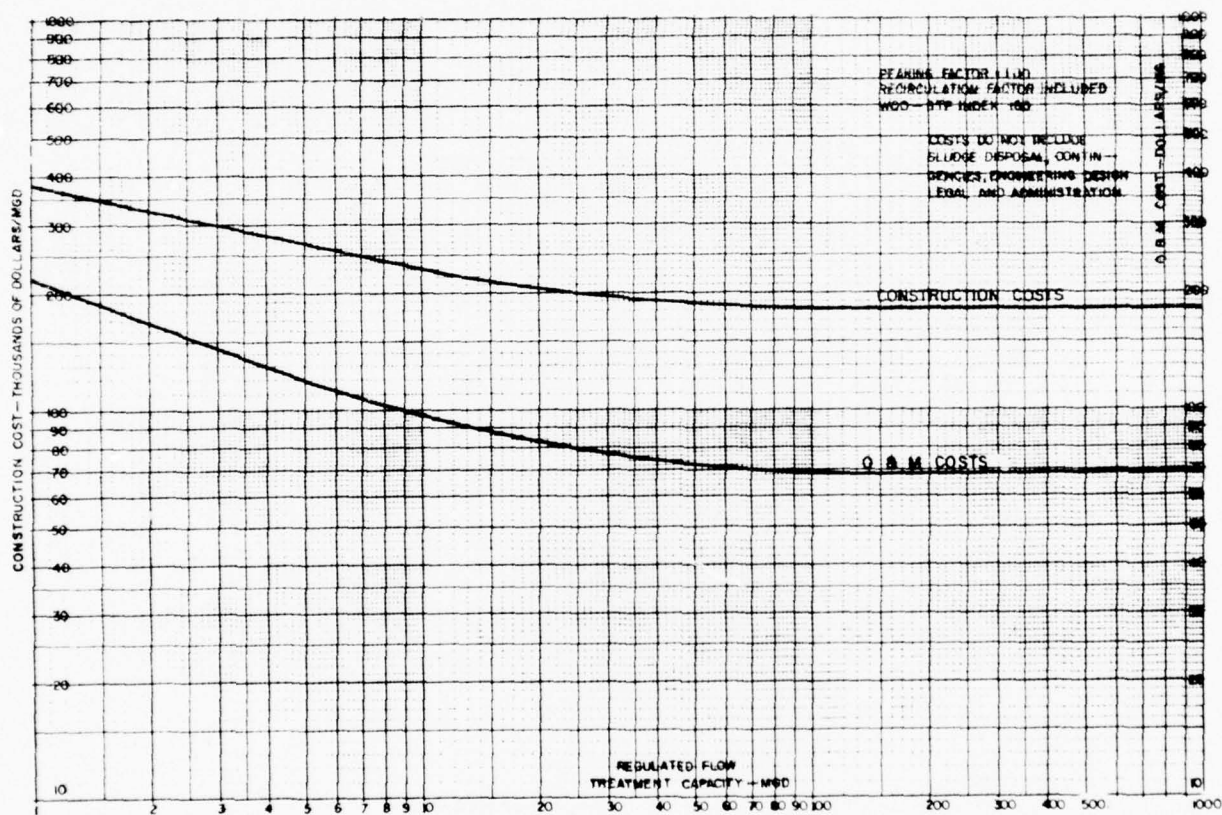
Figure B-VI-A-7

CLINOPTILOLITE ION EXCHANGE
COST CURVE

Total Mixed-Media Filtration O & M Cost
     (Physical-Chemical) = $12/MG

Total Mixed-Media Filtration O & M Cost
     (Advanced Biological) = $11.75/MG

The mixed-media filtration O & M and unit capital costs for plant capacities ranging from 1 - 1,000 MGD are presented in Figure B-VI-A-8.

Chlorination

Labor
0.5 Skilled Labor $ 7,500/year
@4.5 Shifts = $34,000/year
Unit Labor Cost = $1/MG

Chemicals & Supplies
Chlorine @ 4 mg/l & $0.05/pound $1.75/MG
Supplies @ 1% Capital Cost/Year $0.15/MG
                                                     ————————
Total Chemical & Supply Cost $1.90/MG

Energy
Minor Electricity Requirements = $0.10/MG

Total Chlorination O & M Cost = $3/MG

The unit O & M and capital costs for the above mentioned modular chlorination facilities are presented in the cost curves of Figure B-VI-A-9.

Post Aeration

Labor
1 Skilled Labor $15,000/year
@4.5 Shifts = $67,000/year
Unit Labor Cost = $2/MG
        for the advanced biological system.

For the physical-chemical system, the unit labor cost increases to $2.25/MG since this process has a greater share of the general plant function labor cost.

Chemicals & Supplies
Supplies @1% Capital Cost/Year = $0.75/MG
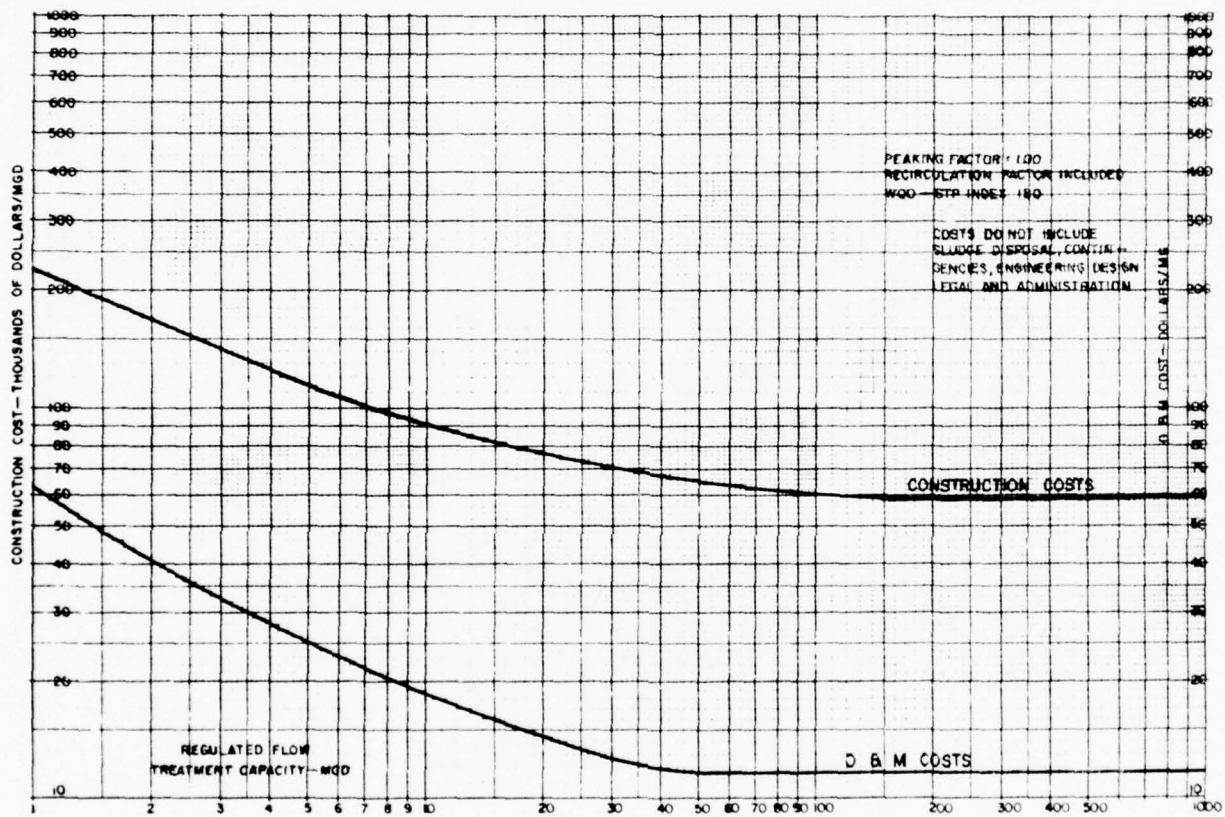
Figure B-VI-A-8

MIXED-MEDIA FILTRATION
COST CURVE
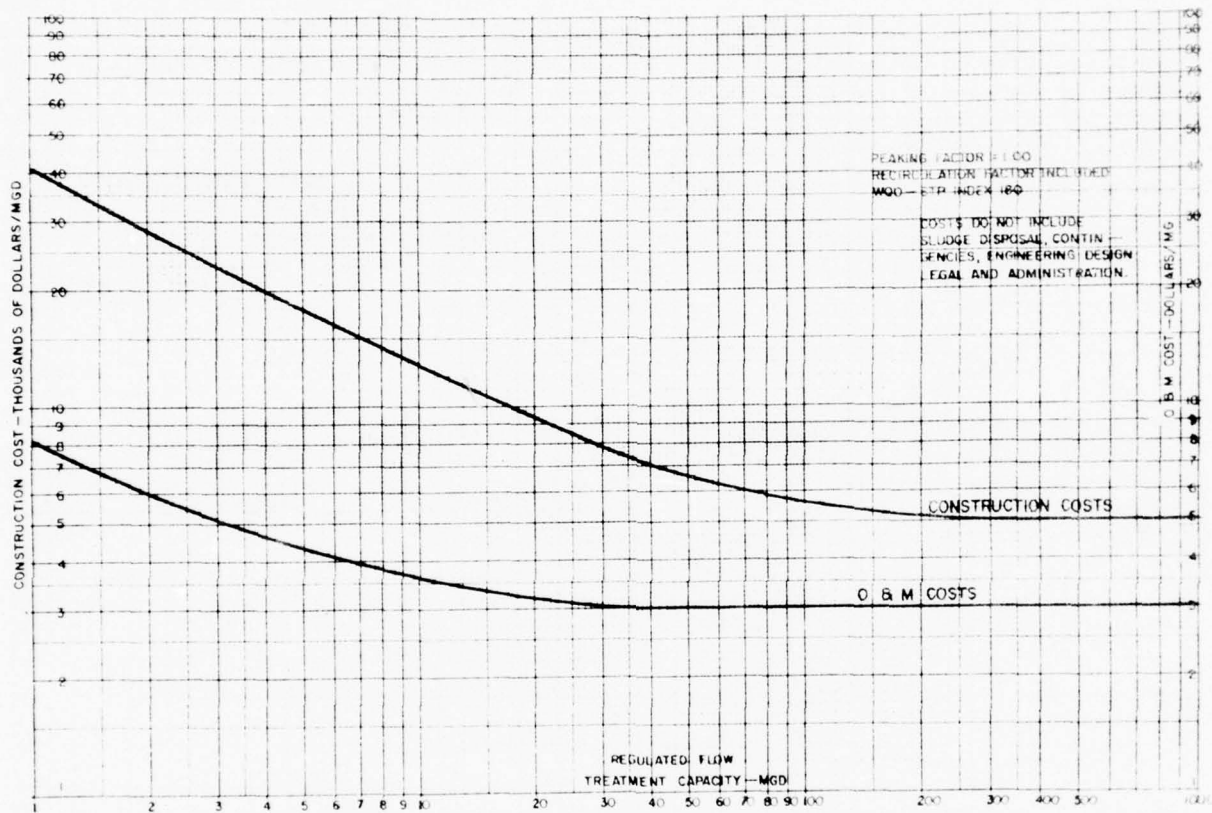
Figure B-VI-9

CHLORINATION
COST CURVE

B-VI-A-24

Energy
Aeration@3,000 HP operating at 2/3 capacity $3.50/MG
& $0.01/KWH =

Total Post-Aeration O & M Cost
(Physical-Chemical) = $6.50/MG

Total Post-Aeration O & M Cost
(Advanced Biological) = $6.25/MG

These unit process costs are presented in the cost cure Figure B-VI-A-10.

Pretreatment

Labor
1 Skilled Labor $15,000/year
0.5 Unskilled Labor $ 5,000/year
     Labor Cost/shift $20,000/year
   @4.5 Shifts = $90,000/year
Unit Labor Cost = $3/MG

Chemicals & Supplies
Supplies @1% Capital Cost/Year = $0.50/MG

Energy
Lift Station & Grit Removal @1,100 HP
     & $0.01/KWH = $2/MG

Total Pretreatment O & M Cost = $5.50/MG

Presented in Figure B-VI-A-11 are the O & M and capital costs for the pretreatment component of the physical-chemical system.

SUMMARY OF TREATMENT PLANT SYSTEM COSTS

Introduction

The advanced biological and physical-chemical treatment plant system capital, operation & maintenance and replacement costs are summarized in this section. The unit costs presented, reflect a modular system capacity of 100 MGD above which no further economies of scale can be projected.
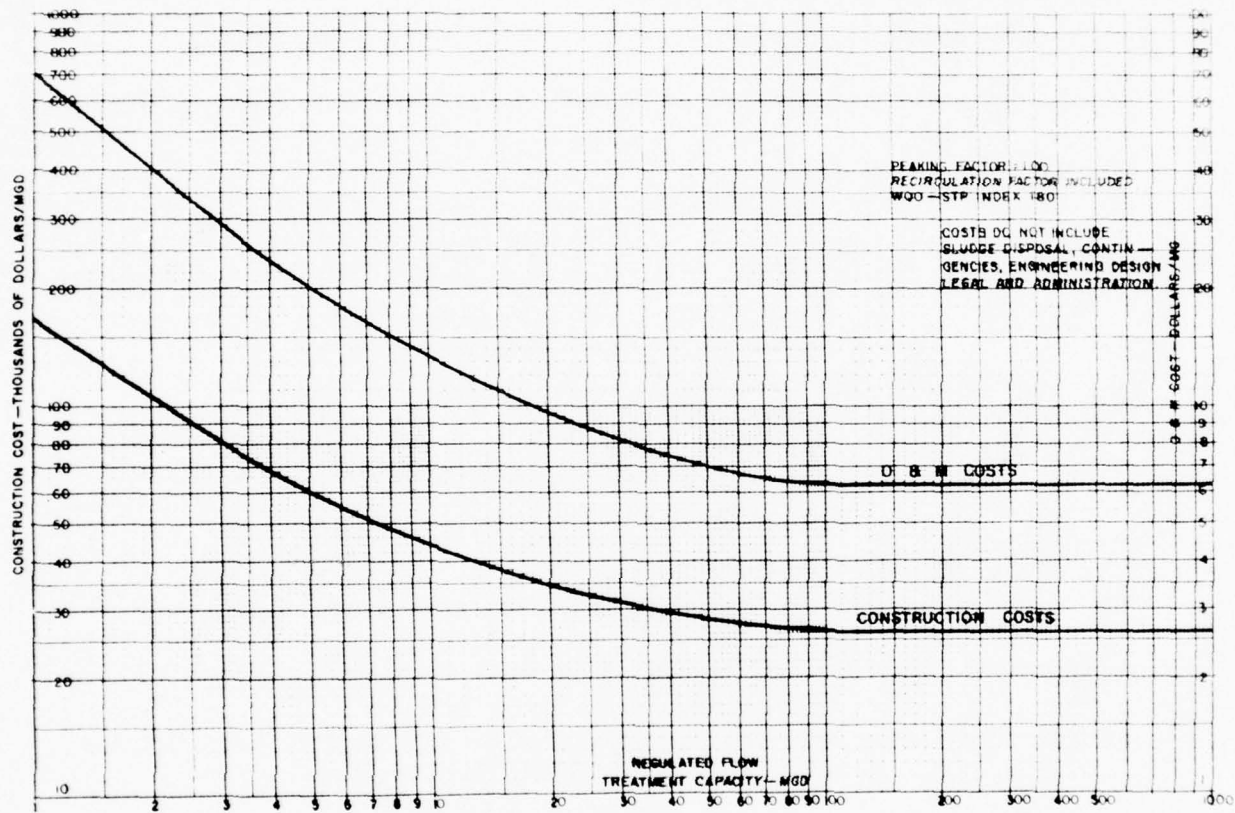
Figure B-VI-A-10

POST AERATION
COST CURVE
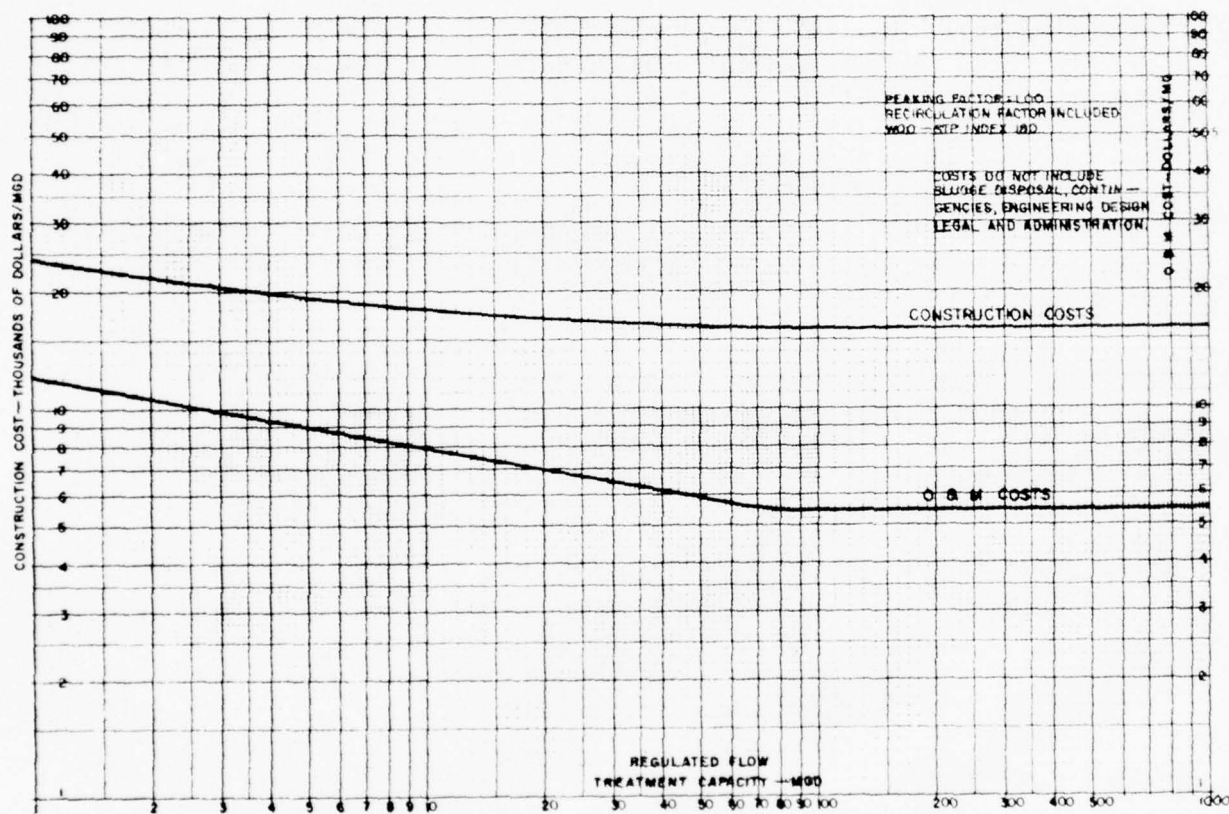
Figure B-VI-A-11

PRETREATMENT
COST CURVE

Capital Costs

A summary of the unit process capital costs which comprise the advanced biological system is presented in Table B-VI-A-1. The unit capital cost for this 100-MGD module is $1,191,000/MGD of regulated flow capacity. The land and landscaping costs, interconnection piping, administration and lab buildings are assumed to equal 10% of the treatment facility cost or $119,000/MGD. A contingency factor of 20% of the above-mentioned capital costs is also included in a capital cost analysis of the treatment systems. For the advanced biological, modular plant this equals $262,000/MGD. Finally an engineering, design, legal and administrative cost equal to 15% of the above capital costs is included. This is equal to some $236,000/MGD for the modular design. The total unit capital cost for the advanced biological treatment plant system is $1,808,000/MGD of regulated flow capacity.

Presented in Table B-VI-A-2 is a summary of the unit process capital costs which comprise the physical-chemical treatment system. The unit capital cost for this module is $697,000/MGD. Cost factors for land et. al., contingencies, and engineering et. al., provisions are identical to those previously mentioned for the advanced biological treatment system. For the 100 MGD capacity physical-chemical plant, land, landscaping, interconnection piping, administration and lab buildings cost equal $70,000/MGD. Likewise the contingency cost of this facility equals $153,000/MGD. Finally the engineering, design, supervision, and administration cost equals $138,000/MGD. Thus the total unit capital cost for the physical-chemical treatment system equals $1,058,000/MGD of regulated flow capacity.

Replacement Costs

The replacement cost schedule for the advanced biological system is presented in Table B-VI-A-3. In this table, replacement costs and their schedules are presented as a percentage of both the unit process capital cost and the total advanced biological treatment system capital cost. For the cost analysis of the AWT systems, the replacement costs are present-worthed at a 5.5% interest rate. When the five-year replacement costs are present-worthed for nine payments, the ten-year costs for four payments and the 25-year cost for one payment during the life of the system, the total advanced biological treatment system replacement cost is equal to $283,000/MGD, or some 24% of the total capital cost for the modular 100-MGD facility. When a contingency

Table B-VI-A-1

CAPITAL COST SUMMARY FOR THE
MODULAR ADVANCED BIOLOGICAL TREATMENT PLANT

Unit Process	Capital Cost (Million Dollars)[a]
Secondary Treatment [b]	39.0
Nitrification-Denitrification	32.0
Lime Clarification	18.2
Carbon Adsorption	21.6
Mixed Media Filtration	5.9
Post Aeration	2.6
TOTAL CAPITAL COST	$ 119.1 Million
UNIT CAPITAL COST	$1.191 Million/MGD

[a] Capital costs do not include land, contingency, engineering, design legal and administration costs.

[b] Secondary treatment includes pretreatment, sludge digestion and storage and chlorination.

Table B-VI-A-2

CAPITAL COST SUMMARY FOR THE
MODULAR PHYSICAL-CHEMICAL TREATMENT PLANT

Unit Process	Capital Cost (Million Dollars)[a]
Pretreatment	1.6
Lime Clarification	18.2
Carbon Adsorption	22.7
Clinoptilolite Ion Exchange	18.2
Mixed Media Filtration	5.9
Chlorination	0.5
Post Aeration	2.6
TOTAL CAPITAL COST	$69.7 Million
UNIT CAPITAL COST	$ 0.697/MGD

[a] Capital costs do not include land, contingency, engineering, design, legal and administration costs.

Table B-VI-A-3

REPLACEMENT COST SCHEDULE FOR

THE ADVANCED BIOLOGICAL TREATMENT SYSTEM

Process	% of Total Capital Cost	% Replacement of Unit Cost			% Replacement of System Cost		
		25 Years	10 Years	5 Years	25 Years	10 Years	5 Years
Secondary Treatment	32.7	0.25	0.075	–	0.0828	0.0245	–
Nitrification-Denitrification	26.8	0.20	0.10	–	0.0536	0.0268	–
Lime Clarification	15.3	–	0.15	0.10	–	0.0229	0.0153
Carbon Adsorption	18.1	0.10	0.075	0.05	0.0181	0.0136	0.0090
Mixed Media Filtration	4.9	0.20	0.10	–	0.0098	0.0049	–
Post Aeration	2.2	0.075	0.25	–	0.0016	0.0055	–
TOTAL	100.0				0.1649	0.0982	0.0243

factor of 20% is added to the above-mentioned costs, the total advanced biological replacement cost is $340,000/MGD of regulated flow capacity.

Presented in Table B-VI-A-4 is the replacement cost schedule for the physical-chemical treatment plant system. Utilizing the same cost analysis as described previously for the advanced biological system, the present-worthed physical-chemical replacement cost is equal to $220,000/MGD or 31.5% of the total capital cost for the 100-MGD treatment facility. The total physical-chemical replacement cost including a 20% contingency is $263,000/MGD of regulated flow capacity.

Operation and Maintenance Costs

A summary of the unit process O & M costs which comprise the advanced biological treatment system is presented in Table B-VI-A-5. These costs are categorized under labor, chemical and supplies and energy headings. For the modular 100-MGD advanced biological plant, the unit O & M cost are presented in Table B-VI-A-5 is $219/MG. A contingency factor of 20% for the O & M cost is also included in the cost analysis of the treatment systems. Thus, the total annual unit O & M cost for the advanced biological treatment system is $263/MG of treated influent.

Presented in Table B-VI-A-6 is a summary of the unit process O & M costs which make up the physical-chemical treatment system. As shown in this table, the unit O & M cost for the 100-MGD modular facility is $218/MG. Including a 20% contingency, the total unit O & M cost for the physical-chemical treatment system is $262/MG of treated influent.

Total Unit Costs

The advanced biological and physical-chemical treatment plant system capital and O & M unit cost curves are presented in Figures B-VI-A-12 and B-VI-A-13, respectively. These curves summarize the modular treatment component unit costs previously developed in this section in order to meet the NDCP effluent standard. The capital or construction cost curve does not include costs for land, contingencies or engineering, design, legal and administration fees. Likewise, the O & M cost curve does not include any provisions for contingency costs. Presented in Figures B-VI-A-14 through B-VI-A-18 are the capital and O & M unit cost curves for treatment technologies which are designed to meet existing effluent quality standards. These cost

Table B-VI-A-4

REPLACEMENT COST SCHEDULE FOR THE PHYSICAL-CHEMICAL TREATMENT SYSTEM

Process	% of Total Capital Cost	% Replacement of Unit Cost			% Replacement of System Cost		
		25 Years	10 Years	5 Years	25 Years	10 Years	5 Years
Pretreatment	2.3	-	0.20	-	-	0.0046	-
Lime Clarification	26.1	-	0.15	0.10	-	0.0392	0.0261
Carbon Adsorption	32.6	0.10	0.075	0.05	0.0326	0.0245	0.0163
Clinoptilolite Ion Exchange	26.1	0.10	0.075	0.05	0.0261	0.0196	0.0131
Mixed Media Filtration	8.5	0.20	0.10	-	0.0017	0.0085	-
Chlorination	0.7	-	0.25	-	-	0.0018	-
Post Aeration	3.7	0.075	0.25	-	0.0028	0.0093	-
TOTAL	100.0				0.0632	0.1075	0.0555

Table B-VI-A-5

OPERATION AND MAINTENANCE COST SUMMARY
FOR THE MODULAR ADVANCED BIOLOGICAL TREATMENT PLANT

Process	O & M Cost[a] (Dollars/MG)			
	Labor	Chemicals & Supplies	Energy	Total O&M Cost
Secondary Treatment	37.00	12.00	8.00	57.00
Nitrification-Denitrification	15.00	13.00	8.00	36.00
Lime Clarification	16.00	18.00	17.00	51.00
Carbon Adsorption	36.00	14.00	7.00	57.00
Mixed Media Filtration	6.50	5.00	0.25	11.75
Post Aeration	2.00	0.75	3.50	6.25
TOTAL O & M COST	111.50	62.75	44.75	219.00

[a] O & M costs do not include any contingency factors.

Table B-VI-A-6

OPERATION AND MAINTENANCE COST SUMMARY
FOR THE MODULAR PHYSICAL-CHEMICAL TREATMENT PLANT

Process	O & M Cost[a] (Dollars/MG)			
	Labor	Chemicals & Supplies	Energy	Total O&M Cost
Pretreatment	3.00	0.50	2.00	5.50
Lime Clarification	17.00	18.00	20.00	55.00
Carbon Absorption	36.00	23.00	8.00	67.00
Clinoptilolite Ion Exchange	23.00	28.00	18.00	69.00
Mixed Media Filtration	6.75	5.00	0.25	12.00
Chlorination	1.00	1.90	0.10	3.00
Post Aeration	2.25	0.75	3.50	6.50
TOTAL O & M COST	89.00	77.15	51.85	218.00

[a] O & M costs do not include any contingency factors
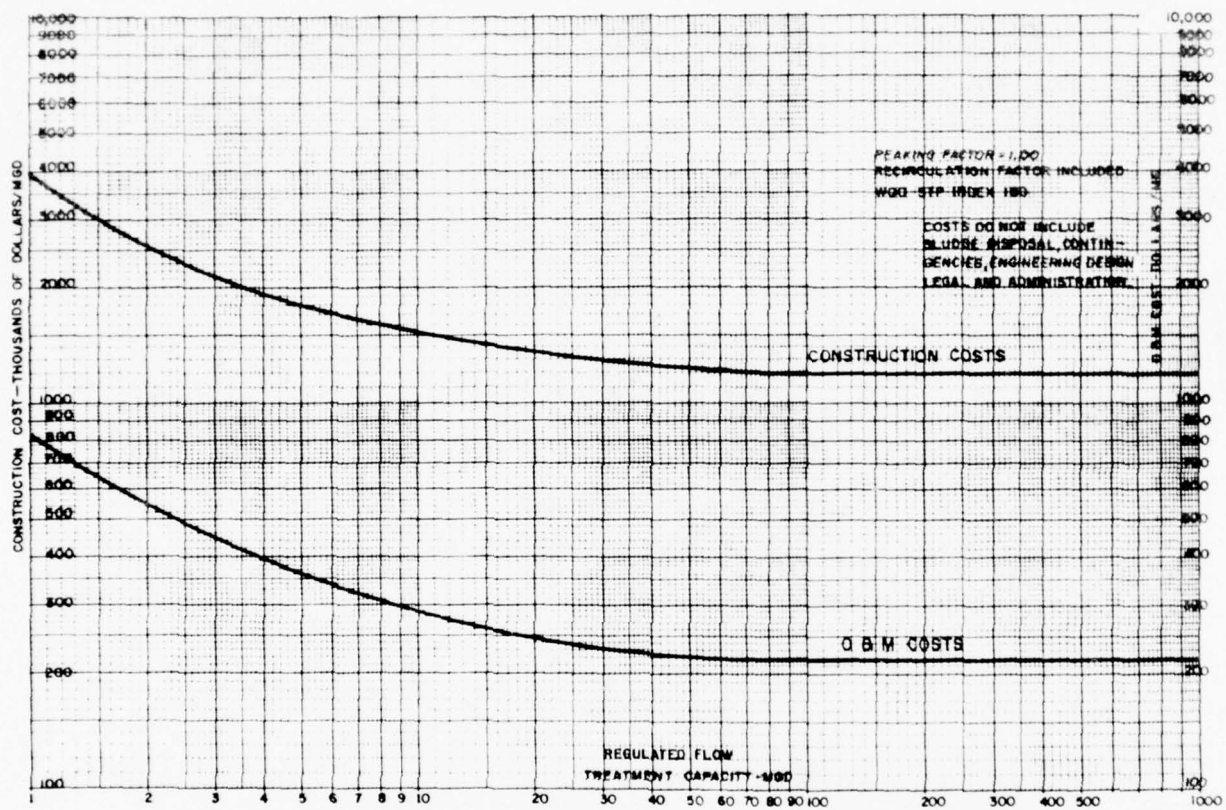
Figure B-VI-A-12

ADVANCED BIOLOGICAL TREATMENT
COST CURVE
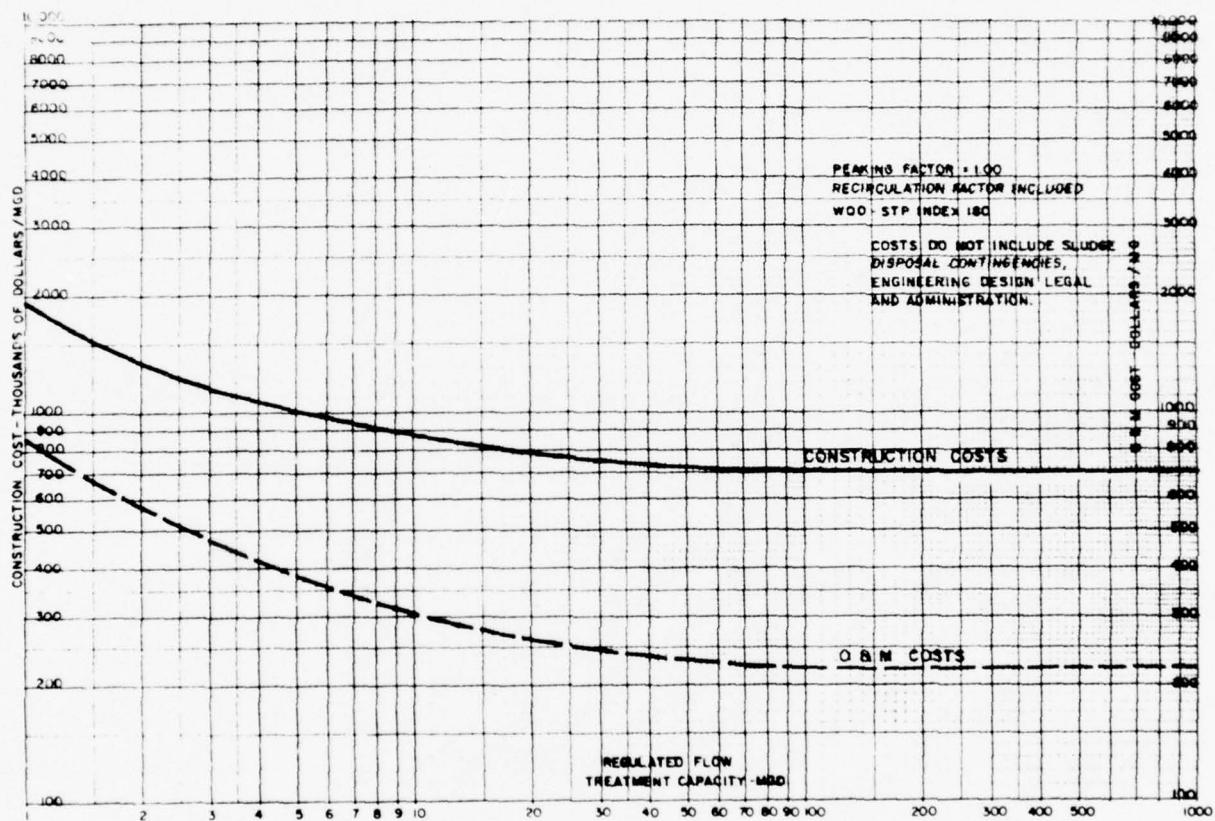
Figure B-VI-A-13

PHYSICAL - CHEMICAL TREATMENT
COST CURVE

B-VI-A-37
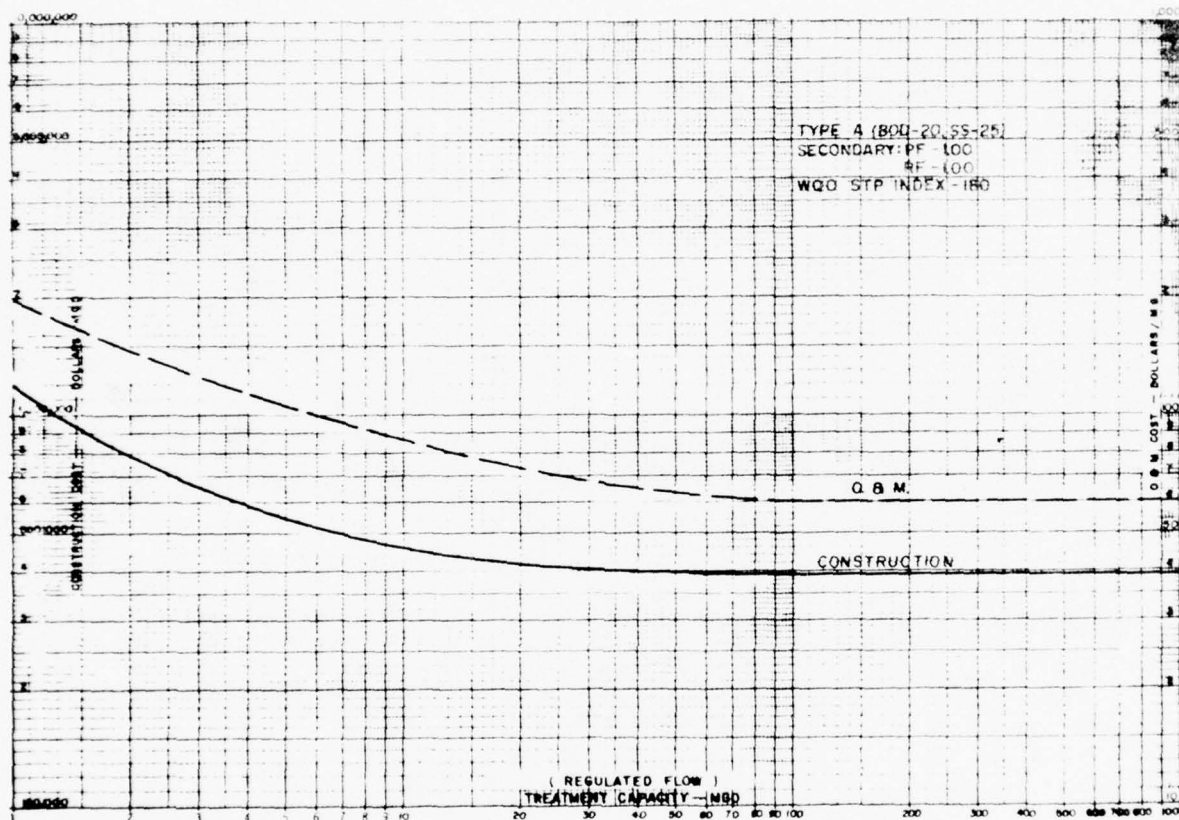
Figure B-VI-A-14

TYPE A-CONVENTIONAL BIOLOGICAL PLUS ADD-ON AWT
FOR PRESENT QUALITY GOALS
COST CURVE

TYPE B (BOD-10, SS-12)
SECONDARY: PF-1.00
RF-1.05
FILTRATION: PF-1.00
RF-1.10
WQO STP INDEX-180

O. & M.

CONSTRUCTION

REGULATED FLOW
TREATMENT CAPACITY — MGD
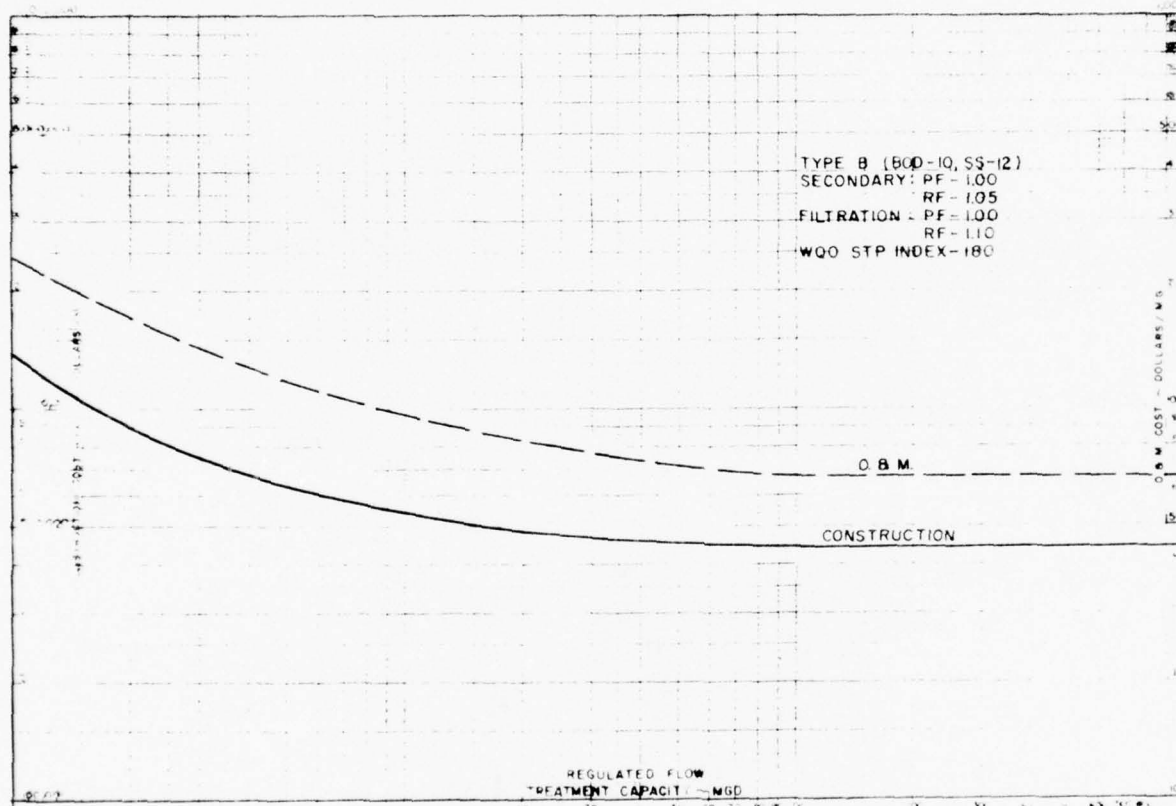
Figure B-VI-A-15

TYPE B-CONVENTIONAL BIOLOGICAL PLUS ADD-ON AWT
FOR PRESENT QUALITY GOALS
COST CURVE

B-VI-A-39
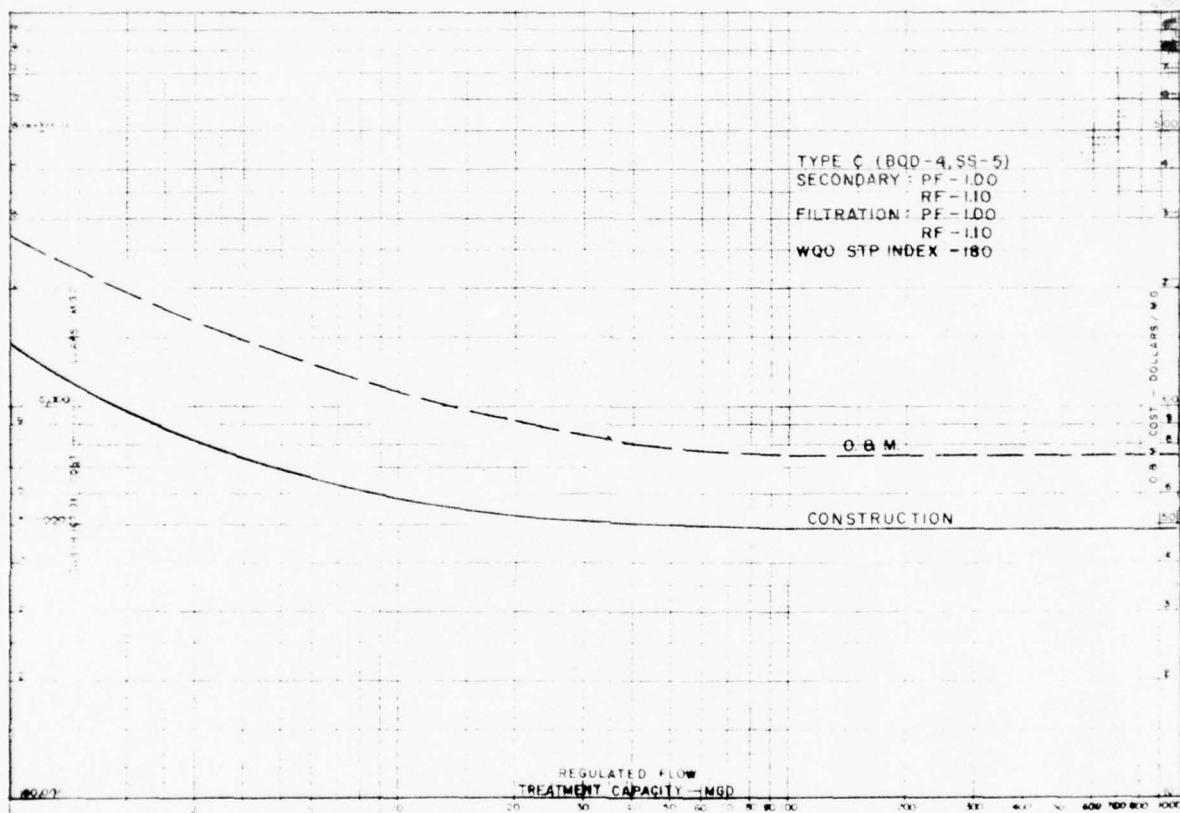
Figure B-VI-A-16

TYPE C-CONVENTIONAL BIOLOGICAL PLUS ADD-ON AWT
FOR PRESENT QUALITY GOALS
COST CURVE
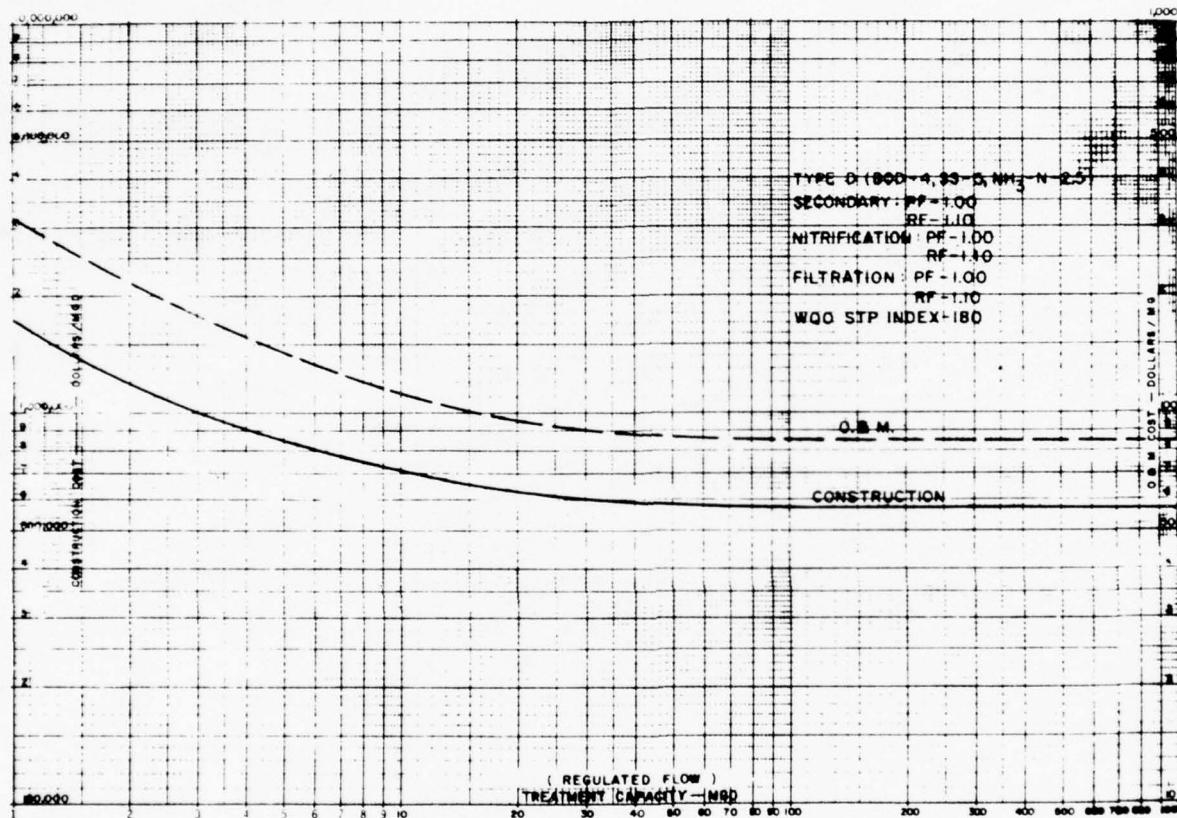
Figure B-VI-A-17

TYPE D-CONVENTIONAL BIOLOGICAL PLUS ADD-ON AWT
FOR PRESENT QUALITY GOALS
COST CURVE
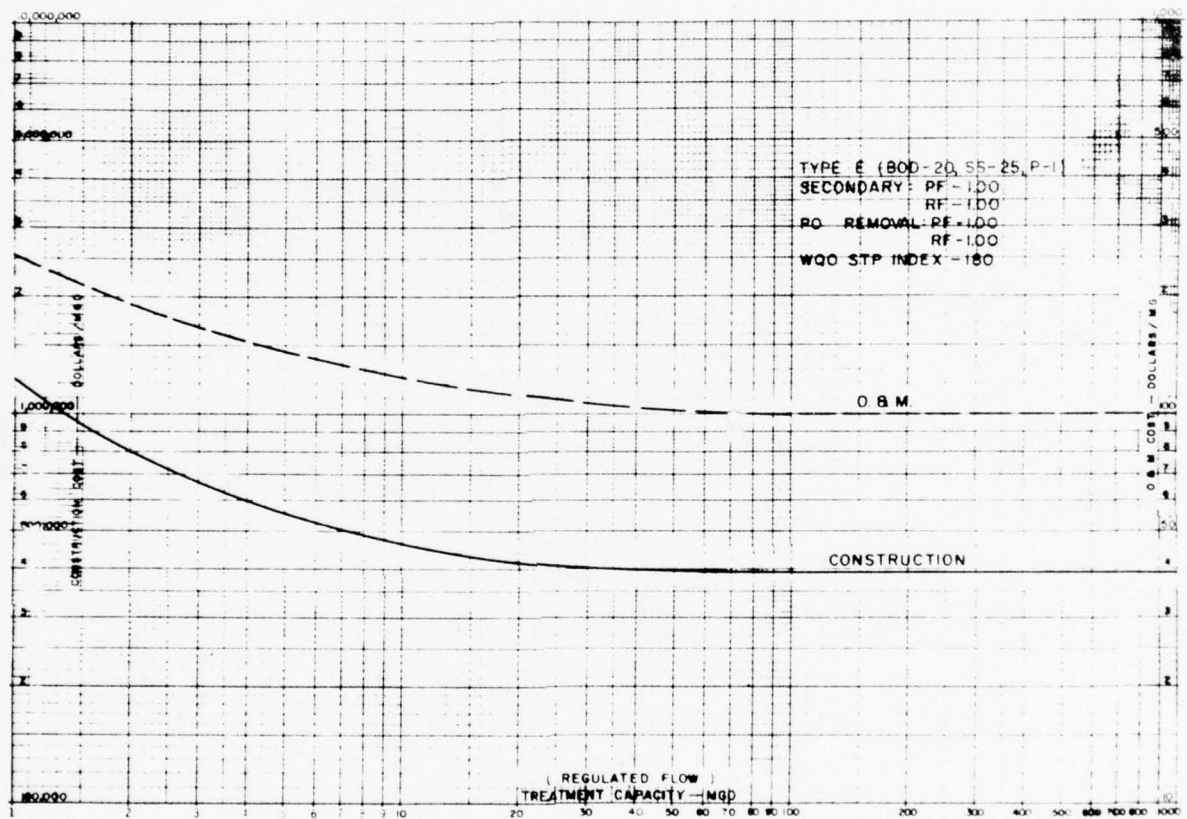
Figure B-VI-A-18

TYPE E-CONVENTIONAL BIOLOGICAL PLUS ADD-ON AWT
FOR PRESENT QUALITY GOALS
COST CURVE

curves were developed from the detailed conventional secondary treatment system plus appropriate advanced waste treatment unit processes presented in this cost section.

Presented in Table B-VI-A-7 is a summary of the AWT system unit costs. These costs reflect provisions for land, contingencies, engineering, design, legal and administration services. It should be noted that the advanced biological system may have decreased capital costs associated with it since it can incorporate existing secondary treatment facilities. Assuming a 100-MGD secondary facility presently exists, the total advanced biological unit capital cost decreases to $1,215,000/MGD of regulated flow capacity.

Table B-VI-A-7

TOTAL UNIT COSTS FOR THE TREATMENT PLANT SYSTEMS

	Unit Costs[a]		
	Capital	Replacement	O & M
Treatment Plant System	(Dollars/MGD)		(Dollars/MG)
Advanced Biological	1,808,000	340,000	263
Physical-Chemical	1,058,000	263,000	262

[a] Costs include provisions for land, contingencies, engineering, design legal and administration services. Sludge disposal costs are not included.

LAND TREATMENT SYSTEM - CAPITAL COSTS

Introduction

The following unit process costs relate to the 265-MGD average daily wastewater influent flow to the land treatment module. These unit process costs do not reflect provisions for sludge disposal, land, contingencies, engineering, design, legal and administration services.

Wastewater Lift Station and Grit Removal

As mentioned in the component basis of design, the module wastewater lift station has a peak pumping capacity of 340 MGD for a lift of 625 feet. The lift station discharges the water to concrete tanks for grit removal prior to biological treatment.

Lift Station Structure & Pumping Facilities @44,000 HP and $175/HP	$ 7.6 Million
Aerated Grit Tanks & Grit Removal Facilities	2.6 "
Total Lift Station & Grit Removal Capital Cost	$10.2 Million

Aerated Lagoon

The capital costs for the aerated lagoons include earthwork for lagoon cell construction, lagoon slope stabilization, pavement construction, flow structures and mechanical surface aerator-mixers. The capital costs for the 265-MGD module are:

Earthwork @$0.65/c.y.	$ 1.9 Million
Slope Stabilization @$6.65/s.y.	1.2 "
Roadway Pavement @$4.00/s.y.	0.1 "
Flow Structures	2.5 "
Aerator-Mixers @$365/HP	5.0 "
Total Aerated Lagoon Capital Cost	$10.7 Million

Storage Facilities

The capital costs for this unit process include site preparation earthwork for lagoon construction, lagoon slope stabilization, pavement construction, flow structures and chlorination facilities. The capital costs for the land treatment module are as follows:

Site Preparation & Clearing	$ 0.9 Million
Earthwork @$0.65/c.y.	8.3 "
Slope Stabilization @$2.00/s.y.	3.1 "
Roadway Pavement @$4.00/s.y.	0.6 "
Flow Structures & Drainage Canal	6.5 "
Chlorination Facilities	2.8 "
Total Storage Lagoon Capital Cost	$20.9 Million

Irrigation System

The capital costs for this unit process include irrigation pumping facilities, a flow distribution network and the irrigation machines. The capacity of this system is 615 MGD which is reflected in the costs.

Pumping Facilities & Conduits	$45.7 Million
Irrigation Machines	7.9 "
Total Irrigation System Capital Costs	$53.6 Million

Drainage System

Drain tile, channel construction, sewer pipe, and drainage tunnels are included in capital costs of this unit process. The capacity of the drainage system is equal to that of the irrigation system - 615 MGD. The capital costs for this system are:

Drain Tile @$220/Ac $^{1/}$	$ 5.8 Million
Sewer Pipe @$590/Ac	15.6 "
Channel Excavation @$120/Ac	3.2 "
Pumping Facilities & Force Main @$16/Ac	4.2 "
Drainage Tunnel @$455/Ac	12.1 "
Total Drainage System Capital Cost	$40.9 Million

$^{1/}$The $220/Ac figure is a weighted average of the two soil types for the C-SELM land treatment areas. For the silt loam type soils, 29,000 feet of four inch diameter plastic drain tile (100 foot spacing) are required to drain the area irrigated by a 1000 foot irrigation machine (72 acres). The drain tile cost for this soil type using a unit pipe cost of $1.12/foot is $450 per acre. For the sandy type soils, 8000 feet of six inch diameter drain tile are required for the drainage of a 72 acre irrigation plot. The drain tile cost for this soil type using a unit pipe cost of $1.42/foot is $160/Ac. Approximately 80% of the irrigation land is classified as sandy soils with the remainder being silt loam soils. Thus the overall drain tile cost is .8 (160) + .2 (450) = $220/Acre.

Miscellaneous Land System Components

The land treatment system costs include electrical facility construction in the rural areas together with building structure costs for administration, maintenance and lab buildings and a reclaimed water monitoring system.

Electrical Facilities	$ 6.2 Million
Buildings	2.3 "
Monitoring System	0.5 "
Total Miscellaneous System Capital Cost	$ 9.0 Million

LAND TREATMENT SYSTEM - REPLACEMENT COSTS

Introduction

The land treatment system replacement costs are programmed capital expenditures for certain treatment components which are to be replaced within the 50 year life of the system. The following replacement costs for the various unit processes of the land treatment system are presented as follows:

Wastewater Lift Station & Grit Removal

For this unit process, 10% of the grit collection and removal facilities are programmed to be replaced every ten years. This is equal to a capital expenditure of $0.3 Million. Also, 50% of the pumping facilities for the main wastewater lift station are replaced every ten years. This is equivalent to a $3.8 Million replacement cost.

Aerated Lagoon

The life of the aerator-mixers is ten years. Therefore 100% of the aerator cost, or $5.0 Million, is programmed to be replaced every ten years.

Storage Facilities

The chlorination facilities are replaced every ten years at a cost equal to 25% of the facilities. For the 265-MGD land treatment module, this is equal to a $0.7 Million capital expenditure for four times during the life of the system.

Irrigation System

The irrigation pumps are replaced every ten years at a cost equal to 80% of the irrigation pump station. For the 265-MGD modular land site, this is equal to $3.1 Million every ten years. Every 15 years, the irrigation machines are to be replaced at a cost equal to 90% of the total capital irrigation machine cost. This is equivalent to three payments during the life of the system equal to $8.6 Million Dollars.

Miscellaneous System Components

Major electrical repairs to the land treatment system are programmed after 25 years of system operation. This replacement cost is equal to 35% of the total electrical facilities cost. This is equal to some 2.2 Million for the modular design land treatment site.

LAND TREATMENT SYSTEM - OPERATION AND MAINTENANCE COSTS

Introduction

The operation and maintenance costs of the treatment facility components include labor, chemicals and supplies and energy requirements. The main wastewater lift station and aerated treatment lagoons are similar to the treatment plant systems in that they require 24 hours maintenance or 4.5 shifts on a year-round basis. The irrigation and drainage systems require eight hours per day maintenance on a year-round basis. During the winter months, when the irrigation machines are not in operation, labor is still utilized for major overhauls of these machines. The eight hour per day maintenance requirement relates to 1.5 shifts when vacation, sick time and weekends are taken into account. The unit labor costs and labor categories are identical to those previously presented in the treatment plant system section. The following O & M costs are presented for the major process units.

Labor

Main wastewater lift station & grit removal

1 Supervisor	$ 20,000/year
4 Skilled Labor	60,000/year
1.5 Unskilled Labor	15,000/year
Labor Cost/shift	$ 95,000/year
@4.5 Shifts	$428,000/year

General plant functions

5 Supervisors	$100,000/year
4 Unskilled Labor	40,000/year
Labor Cost	$140,000/year

Aerated lagoon

1 Supervisor	$ 20,000/year
11 Skilled Labor	165,000/year
Labor Cost/shift	$185,000/year
@4.5 Shifts	$833,000/year

Storage lagoon facilities

1 Supervisor	$ 20,000/year
14 Unskilled Labor	140,000/year
Labor Cost/shift	$160,000/year
@1.5 Shifts	$240,000/year

Chlorination facilities

3 Skilled Labor	$ 45,000/year
@4.5 Shifts	$202,000/year

Irrigation & drainage inspection & monitoring

2 Supervisors	$ 40,000/year
4 Skilled Labor	60,000/year
30 Unskilled Labor	300,000/year
Labor Cost/shift	$400,000/year
@1.5 Shifts	$600,000/year

Irrigation & drainage system maintenance

1 Supervisor	$ 20,000/year
4 Skilled Labor	60,000/year
4 Unskilled Labor	40,000/year
Labor Cost/shift	$120,000/year
@1.5 Shifts	$180,000/year

Chemicals & Supplies

Chlorine @4 mg/l & $0.05/pound	=	$170,000/year
Main lift station @1% Capital Cost/Year	=	76,000/year
Aerator-Mixers @1% Capital Cost/Year	=	50,000/year
Chlorination facilities @1% Capital Cost/Yr.	=	28,000/year
Irrigation pumps @1% Capital Cost/Year	=	38,000/year
Irrigation machines @1% Capital Cost/Year	=	79,000/year
Transmission facilities @0.1% Capital Cost/Year	=	86,000/year

Energy

For the land treatment system, all energy costs reflect electricity requirements for the various components presented herein. These costs are based on a electricity rate of $0.01/KWH.

Main wastewater lift station @44,000 HP	=	$2,234,000/year
Aerated lagoon @13,500 HP	=	878,000/year
Irrigation distribution system @38,500 HP & 158 days in operation	=	1,083,000/year
Irrigation machines @7,100 HP and 158 days in operation	=	198,000/year
Drainage system @14,000 HP and 158 days in operation	=	390,000/year

LAND TREATMENT SYSTEM - LAND COSTS & ANNUAL PAYMENTS

Land Costs

Associated with the land treatment system are a number of land costs and annual land payments. For the land treatment system, the only land that is purchased is for the lagoon facilities. The cost of land, together with relocation costs for families or buildings on this land, are developed on a per acre basis as discussed in detail in the land displacement subsection of Section VII A of this Appendix. For the modular land treatment system, some 5,600 acres of lagoon land are required to accommodate the 265-MGD average daily flow. The unit land and relocation cost for this module is $1,265/acre. Including a 20% contingency cost factor and a 15% engineering, design, legal and administration fee, the total land cost for the land treatment system module is some $9.8 Million or $37,000/MGD of treatment capacity.

Also associated with the land treatment system are initial and inconvenience payments to the participating landowners in the amount of 10% of the present land value within the irrigation system. This payment is used to help the participating farmer defray the cost of new agricultural equipment and also to pay for any loss in crop revenue due to construction of the land treatment system. Based on a gross irrigation land requirement of 66,000 acres for this module, an average land value of $745/acre and a contingency cost factor of 20%, the initial and and inconvenience land payments equal $5.9 Million or $19,000/MGD for this modular land treatment system.

Finally, an initial land cost is paid to people residing within the site boundary who presently utilize shallow wells as a water supply source. The cost includes provisions for constructing deeper wells (200 feet) to replace existing shallow ones so that the rural communities' water supply is isolated from the potable, reclaimed land treatment effluent which interfaces with the groundwater supply. This well cost, including contingencies, equals $2,000 per unit or some $6,000/MGD for the modular design.

Annual Payments

Included in the land cost analysis for the land treatment system is a recognition of the fact that purchased lagoon facilities remove lands from the tax base and hence create an annual tax loss. Based on the modular design requirement of 5,600 acres of lagoon land, an average land value of $745/acre, an average rural tax multiplier of $2/$100 of assessed valuation and a contingency factor of 20%, some $100,000 per year of tax revenue will be lost through construction of the modular facility. In order to make up for this annual tax loss, a unit tax re-

venue treatment cost of $1/MG of treated influent is assessed for the modular design.

Also an annual land cost payment is paid to the participating landowner since his land will be unavailable for other uses during the 50-year life of the treatment system. This annual cost, which is also based on the gross irrigation area utilized by the system, is equal to 4% of the present land value. Based on the modular land system requirement of 66,000 acres, an average land value of $745/acre and a contingency cost factor of 20%, the annual land payment is equal to $2.4 Million/year or $25/MG of treated influent.

SUMMARY OF LAND TREATMENT SYSTEM COSTS

The land treatment system capital, operation & maintenance and replacement costs are summarized in this section for the 265 MGD land treatment module.

Capital Costs

A summary of the component capital costs which comprise the land treatment system is presented in Table B-VI-A-8. The unit capital cost for this 265-MGD module is $550,000/MGD of average daily wastewater flow. For the land treatment system cost analysis, a contingency factor of 20% of the above-mentioned capital cost is included. This is equal to $110,000/MGD for the modular system. An engineering, design, legal and administration cost equal to 15% of the above mentioned capital costs is also included in the cost analysis. For the modular design, this is equal to $99,000/MGD. Thus the total unit capital cost for the land treatment system is $759,000/MGD of average daily wastewater flow.

Replacement Costs

The replacement cost schedule for the land treatment system is presented in Table B-VI-A-9. For the cost analysis of the AWT system, the replacement costs are present-worthed at a 5.5% interest rate. When the ten-year replacement costs are present-worthed for four payments, the 15-year costs for three payments and the 25-year cost for one payment during the life of the system, the total land treatment system replacement cost is equal to $22.9 Million. Including a 20% contingency factor, the total unit replacement cost for the land treatment system is $104,000/MGD of treatment capacity.

Operation and Maintenance Costs

A summary of the component labor costs for the 265-MGD module is presented in Table B-VI-A-10.

Table B-VI-A-8

CAPITAL COST SUMMARY FOR THE
MODULAR LAND TREATMENT SYSTEM

Component	Capital Cost (Million Dollars)[a]
Lift Station & Grit Removal	10.2
Aerated Lagoon	10.7
Storage Facilities	20.9
Irrigation System	53.6
Drainage System	40.9
Miscellaneous Components	8.5
TOTAL CAPITAL COST	$145.3 Million
UNIT CAPITAL COST	$550,000 / MGD

[a] Capital costs do not include land, contingency, engineering, design legal and administration costs.

Table B-VI-A-9

REPLACEMENT COST SCHEDULE
FOR THE LAND TREATMENT SYSTEM

Land System Component	Modular Replacement Cost ($Million)		
	25 years	15 years	10 years
Lift Station	-	-	3.8
Grit Facilities	-	-	0.3
Aeration Facilities	-	-	5.0
Chlorination Facilities	-	-	0.7
Irrigation System	-	8.6	3.1
Electrical System	2.2	-	-
TOTAL	2.2	8.6	12.9

Table B-VI-A-10

SUMMARY OF LAND TREATMENT SYSTEM LABOR COSTS

Components	Total Annual Labor Cost $ (Dollars/Year)
General Plant Functions	$ 140,000
Main Wastewater Lift Station & Grit Removal	428,000
Aerated Lagoon	833,000
Storage Facilities	240,000
Chlorination Facilities	202,000
Irrigation & Drainage Systems	780,000
TOTAL SYSTEM LABOR COST	$2,623,000/Year
UNIT LAND TREATMENT SYSTEM LABOR COST	$27/MG

Presented in Table B-VI-A-11 are the land treatment component chemical and supply costs for the 265-MGD modular facility.

A summary of the component energy costs for the 265 MGD land treatment module is presented in Table B-VI-A-12.

A summary of the 265-MGD land treatment module total O & M unit costs is presented in Table V-VI-A-13. As shown in this table, the unit land treatment O & M cost is $82/MG. The total unit annual O & M cost for the land treatment system including a 20% contingency factor is $98/MG of treated influent.

Land Costs and Annual Payments

A summary of the unit land costs associated with the construction and operation of the 265-MGD land treatment module is presented in Table B-VI-A-14.

Total Land Treatment System Unit Costs

The land treatment system capital and O & M unit cost curves are presented in Figure B-VI-A-19. These curves summarize the modular treatment component unit costs as developed in this section.

Presented in Table B-VI-A-15 is a summary of the land treatment system unit costs for the achievement of the NDCP effluent standard.

PRESENT INDEBTEDNESS FOR EXISTING WASTEWATER TREATMENT FACILITIES

Introduction

A cost analysis of proposed wastewater management treatment systems should take into account the present bonded indebtedness of existing municipal treatment systems within the study area. The data presented in this section is obtained from Appendix F of this report, from personal communication with wastewater management personnel and from an estimation procedure described below.

Indebtedness Estimation Methodology

In cases where indebtedness data is incomplete or not available, an estimation procedure is developed to determine the total system indebtedness (including collection and conveyance) and the treatment facility indebtedness.

Table B-VI-A-11

SUMMARY OF LAND TREATMENT SYSTEM
CHEMICAL AND SUPPLY COSTS

Component	Total Annual Chemical & Supply Cost (Dollars/Year)
Chlorine	$170,000
Main Wastewater Lift Station	76,000
Aeration Facilities	50,000
Chlorination Facilities	28,000
Irrigation & Drainage Facilities	203,000
TOTAL SYSTEM CHEMICAL & SUPPLY COST	$527,000/Year
UNIT LAND TREATMENT SYSTEM CHEMICAL & SUPPLY COST	$5.50/MG

Table B-VI-A-12

SUMMARY OF THE LAND TREATMENT SYSTEM
ENERGY COSTS

Component	Total Annual Energy Cost (Dollars/Year)
Main Wastewater Lift Station	$2,234,000
Aerated Lagoon	878,000
Irrigation System	1,281,000
Drainage System	390,000
TOTAL SYSTEM ENERGY COST	$4,783,000/Year
UNIT LAND TREATMENT SYSTEM ENERGY COST	$49.50/MG

Table B-VI-A-13

OPERATION AND MAINTENANCE COST SUMMARY
FOR THE MODULAR LAND TREATMENT SYSTEM

Cost Category	O & M Unit Cost[a] (Dollars/MG)
Labor	$ 27.00
Chemicals & Supplies	5.50
Energy	49.50
TOTAL O & M COST	$ 82.00/MG

[a]O & M costs do not include any contingency factors.

Table B-VI-A-14

SUMMARY OF LAND TREATMENT SYSTEM
LAND COSTS AND LAND PAYMENTS

Cost Component	Unit Land Cost[a] (Dollars/MGD)	Unit Annual Payment[b] (Dollars/MG)
Land & Relocation Costs	37,000	-
Initial & Inconvenience Payment	19,000	-
Private Well Construction	6,000	-
Annual Land Payment	-	25
Tax Revenue Replacement	-	1
TOTAL UNIT LAND COST PAYMENTS (without tax) (with tax)	$62,000/MGD $62,000/MGD	$25/MG $26/MG

[a]Cost includes a 20% contingency factor together with 15% engineering, design, legal and administration fee.

[b]Payment includes a 20% contingency factor.
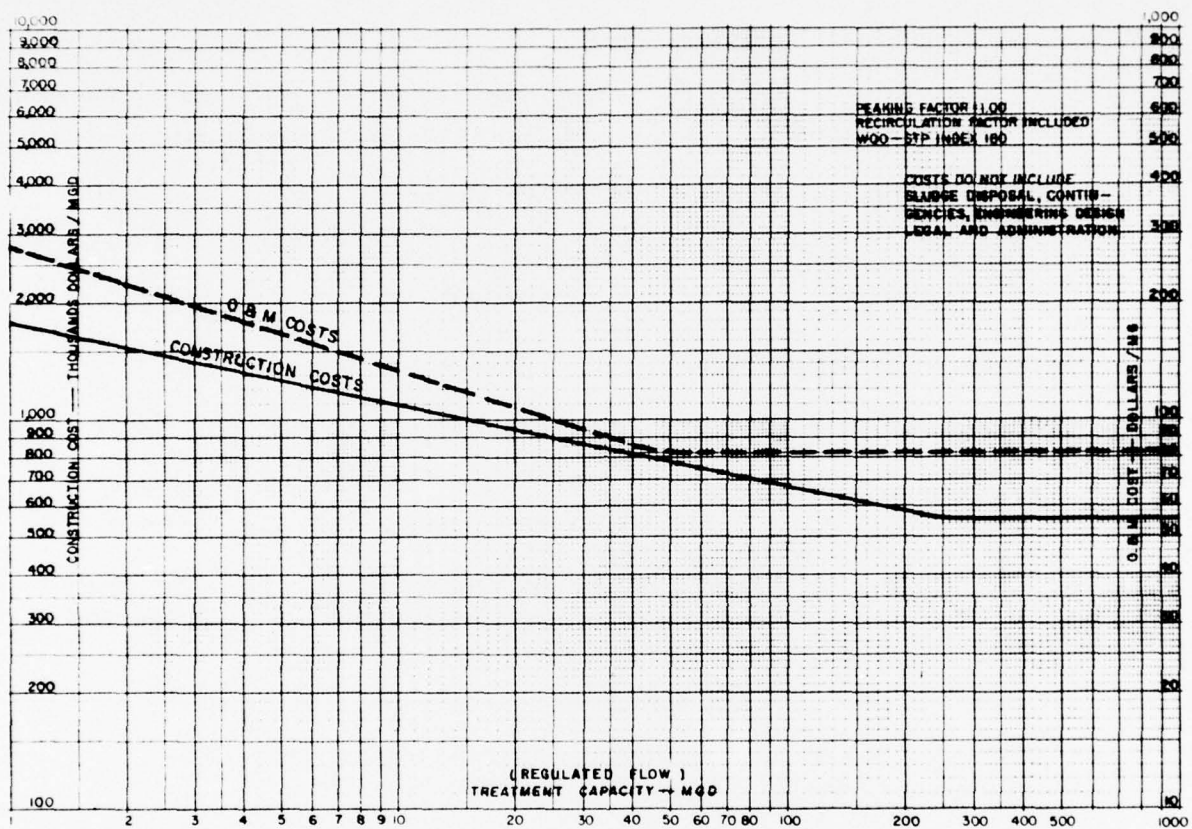
Figure B-VI-A-19

LAND TREATMENT
COST CURVE

Table B-VI-A-15

TOTAL UNIT COSTS FOR THE LAND TREATMENT SYSTEM

| System Component | Unit Costs[a] | | O & M |
| | Capital | Replacement | |
	(Dollars/MGD)		(Dollars/MG)
Treatment Facility	$759,000	$104,000	$ 98
Land Costs & Payments	62,000	-	-
- without tax	-	-	25
- with tax	62,000	-	26
TOTAL LAND TREATMENT SYSTEM UNIT COSTS	$821,000/MGD	$104,000/MGD	-
- without tax	-	-	$123/MG
- with tax	$821,000/MGD	$104,000/MGD	$124/MG

[a]Costs include provisions for land, contingencies, engineering, design legal and administration services. Sludge disposal costs are not included.

For several instances where complete indebtedness information is available, the treatment facility portion represents some two-thirds of the total indebtedness. Thus when only total system indebtedness information is available, it is estimated that the treatment facility indebtedness equals two-thirds of this total.

For the many small wastewater systems where indebtedness is not known, a service population model is utilized to estimate the indebtedness. This model results in a total bonded indebtedness factor of $400 per person serviced by the treatment system. Presented in Tables B-VI-A-16 and B-VI-A-17 are the wastewater management system's outstanding bonded indebtedness for the C-SELM study area in the States of Illinois and Indiana, respectively.

As shown in these tables, the estimated existing wastewater management system bonded indebtedness approximates $615 Million.

BIBLIOGRAPHY B-VI-A

1.   Office of the Chief of Engineers, Department of the Army, Cost Data Annex to the Technical Appendix, Regional Wastewater Management Systems for the Chicago Metropolitan Area, March 1972.

Table B-VI-A-16

OUTSTANDING BONDED INDEBTEDNESS FOR
WASTEWATER TREATMENT PLANTS IN ILLINOIS

Agency	Total Bonded Indebtedness (1,000,000's)	Plant Bonded Indebtedness (1,000,000's)	Population Served (1,000's)
Metropolitan Sanitary District	204.0	136.0[a]	5,497.0
North Shore Sanitary District	35.0	17.5	147.6
DuPage County Department of Public Works	10.5	7.0	27.8
Lake County Department of Public Works	10.4	7.0[a]	20.0
Bloom Township Sanitary District	2.8	2.8	80.0
Remaining Illinois Facilities	253.0[a]	169.0[a]	631.8[a]
	$515.7[a]	$339.3[a]	6,404.2[a]

[a]Estimated figures

Table B-VI-A-17

OUTSTANDING BONDED INDEBTEDNESS FOR
WASTEWATER TREATMENT PLANTS IN INDIANA

Agency	Total Bonded Indebtedness (1,000,000's)	Plant Bonded Indebtedness (1,000,000's)	Population Served (1,000's)
East Chicago Sanitary District	23.2	17.0	57
Hammond Sanitary District	19.0	12.7[a]	180
Gary Sanitary District	18.0	12.0[a]	200
Michigan City Sanitary District	10.1	1.6	63
Portage Sanitary Board	6.8	4.5[a]	20
Valporaiso Sanitation and Sewer Department	1.7	1.2[a]	20
Chesterton Sewage Utility	0.6	0.4[a]	5
Hobart, Crown Point, Dyer, Schererville, Lowell, Hebron, Kouts	19.2[a]	12.8[a]	48[a]
	$98.6[a]	$62.2[a]	593[a]

[a]Estimated figures

B-VI-A-64

VI.   COMPONENT BASIS OF COST

B.   INDUSTRIAL TREATMENT SYSTEMS

INTRODUCTION

Detailed analyses of treatment systems for the critical industries are made in Appendix B, Section IV B.   Detailed costs were obtained from these analyses and are presented in this section.

The purposes of this cost analysis is to identify the incremental costs for the critical C-SELM industries for progressing from treatment technology yielding existing effluent quality standards to treatment technology producing NDCP standards.   The unit costs for the critical industries then will be used to model the costs for the non-critical C-SELM industries.   As a result, the costs for attaining NDCP standards for all existing surface dischargers will be identified.   The industries tributary to municipal treatment plants already pay the cost for NDCP treatment through a system of service charges.   This, the total cost to the C-SELM region for NDCP level treatment can now be established.

COST TO STEEL INDUSTRY

To meet NDCP standards an industry may go directly to an advanced treatment regional plant from its present treatment facility by adding, on site, the processes necessary for maximum recycle.   Another alternative is to proceed to NDCP treatment from facilities designed to meet current standards.   These two alternatives provide 8 paths by which to attain NDCP treatment.

The following tables B-VI-B-1 through 10 contain detailed estimates of costs for steel industry wastewater treatment per module, varying from present practice through NDCP standards.   Table B-VI-B-11 summarizes the total annual cost per module for each path contained in the tables listed above.

The different paths to NDCP treatment take into account cost considerations which stem from the economy of scale of treatment at a regional plant as opposed to treatment "on-site".   Operation and maintenance costs were appropriately altered for a waste stream that has been reduced in volume due to recirculation.   Existing treatment facilities, which would be retired from use when more intensive treatment became necessary, have been included in the cost summaries as expenditures of capital with a yearly amortization charge.

ANNUAL COST SUMMARY OF PRESENT TREAT
1972 DOLLARS

Process & Treatment	Capital Cost	Amortizati
Plain sedimentation of flue gas wash water from blast 1/ & steel furnace & sinter plant (sludge handling included)	4,500,000	265,
Plain sedimentation of scale bearing waters from hot rolling mills (sludge handling included)1/	3,750,000	221,
Coagulation & sedimentation of flood lubricating emulsions in cold mills1/	4,500,000	265,
Biological treatment of sanitary & by-product coke plant wastes in aerated lagoons (sludge handling included)1/	790,000	47,
Pickle liquor & rinse neutralization with sludge lagooning1/	1,875,000	110,
Ion exchange for chrome from plating rinse operation1/	216,000	13,
TOTALS	15,631,000	921,

TABLE B-VI-B-1

SUMMARY OF PRESENT TREATMENT FACILITIES
1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
4,500,000	265,000	90,000	1,210,000	-675,000	890,000
3,750,000	221,000	75,000	378,000	-1,310,000	-636,000
4,500,000	265,000	90,000	140,000	- 67,000	428,000
790,000	47,000	16,000	1,465,000	-	1,528,000
1,875,000	110,000	38,000	1,270,000	-	1,418,000
216,000	13,000	4,000	70,000	-	87,000
15,631,000	921,000	313,000	4,533,000	-2,052,000	3,715,000

B-VI-B-2

Process & Treatment	Capital Cost	Amortizati
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	71,
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,443,000	85,
Thermal regeneration of pickle liquor [6]	790,000	46,
Ion exchange for chromium plating wastes [1]	216,000	13,
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110,
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47,
Carbon columns (biological operation) [2]	2,666,000	157
80% Phosphorus removal [2]	86,000	5,
Post aeration [2]	508,000	30
TOTAL	24,872,000	1,46

TABLE B-VI-B-2

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
5,400,000	318,000	108,000	1,210,000	-675,000	961,000
4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
1,200,000	71,000	24,000	73,000	–	168,000
1,441,000	85,000	28,000	14,000	–	127,000
790,000	46,000	16,000	217,000	-830,000	-551,000
216,000	13,000	4,000	70,000	–	87,000
1,875,000	110,000				110,000
5,400,000	319,000	108,000	140,000	- 67,000	500,000
790,000	47,000	16,000	1,465,000	–	1,528,000
2,666,000	157,000	53,000	263,000	–	473,000
86,000	5,000	2,000	181,000	–	188,000
508,000	30,000	10,000	54,000	–	94,000
24,872,000	1,466,000	459,000	4,065,000	-2,882,000	3,108,000

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B-VI-B-3

ANNUAL COST SUMMARY OF ON-SITE ADVANCED BIOLOGICAL TREATMENT (NDCP) VI

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
Plain sedimentation & recirculation of flue gas wash water [1]/	5,400,000	318,0
Plain sedimentation & recirculation of scale breaking water [1]/	4,500,000	265,0
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]/	1,200,000	71,0
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]/	1,44?,000	85,0
Thermal regeneration of pickle liquor [6]/	790,000	46,0
Ion exchange for chromium plating wastes [1]/	216,000	13,0
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]/	1,875,000	110,0
Palm oil recovery from flood lubricating wastewater from cold mill [1]/	5,400,000	319,0
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon with sludge handling) [1]/ [2]/	790,000	47,0
Carbon columns (biological operation) [2]/	2,666,000	157,0
98% Phosphorus removal (lime clarification - biological) [2]/	3,261,000	192,0
Nitrification-Denitrification [2]/	5,580,000	329,0
Final filtration [2]/	1,066,000	63,0
Post aeration [2]/	508,000	30,0
TOTAL	34,693,000	2,045,

TABLE B-VI-B-3

BIOLOGICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
	5,400,000	318,000	108,000	1,210,000	-675,000	961,000
	4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
	1,200,000	71,000	24,000	73,000	-	168,000
ary	1,443,000	85,000	28,000	14,000	-	127,000
	790,000	46,000	16,000	217,000	-830,000	-551,000
	216,000	13,000	4,000	70,000	-	87,000
	1,875,000	110,000				110,000
1/ mill	5,400,000	319,000	108,000	140,000	- 67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
2/	2,666,000	157,000	53,000	263,000		473,000
	3,261,000	192,000	65,000	308,000		565,000
	5,580,000	329,000	112,000	190,000		631,000
	1,066,000	63,000	21,000	77,000		161,000
	508,000	30,000	10,000	54,000		94,000
	34,693,000	2,045,000	655,000	4,459,000	-2,882,000	4,277,000

B-VI-B-4

ANNUAL COST SUMMARY OF ON-SITE PYSICAL-CHEMICAL TREATMENT (NDCP) V

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,000
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,000
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	71,000
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,441,000	85,000
Thermal regeneration of pickle liquor [6]	790,000	46,000
Ion exchange for chromium plating wastes [1]	216,000	13,000
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110,000
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,000
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47,000
Carbon columns (biological operation) [2]	2,666,000	157,000
98% Phosphorus removal (lime clarification - biological) [2]	3,261,000	192,000
Nitrogen removal with clinoptilolite [2]	2,728,000	161,000
Final filtration [2]	1,066,000	63,000
Post aeration [2]	508,000	30,000
TOTAL	31,841,000	1,877,000

TABLE B-VI-B-4

CAL-CHEMICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
	5,400,000	318,000	108,000	1,210,000	-675,000	961,000
	4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
	1,200,000	71,000	24,000	73,000	-	168,000
	1,441,000	85,000	28,000	14,000	-	127,000
	790,000	46,000	16,000	217,000	-830,000	-500,000
	216,000	13,000	4,000	70,000	-	87,000
	1,875,000	110,000				110,000
1/ ill	5,400,000	319,000	108,000	140,000	- 67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
	2,666,000	157,000	53,000	263,000		473,000
	3,261,000	192,000	65,000	308,000		565,000
	2,728,000	161,000	55,000	416,000		632,000
	1,066,000	63,000	21,000	77,000		161,000
	508,000	30,000	10,000	54,000		94,000
	31,841,000	1,877,000	598,000	4,685,000	-2,882,000	4,278,000

B-VI-B-5

ANNUAL COST SUMMARY OF REGIONAL ADVANCED BIOLOGICAL TREATMENT(NDC

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,000
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,000
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided)[4]	1,200,000	71,000
Cooling towers for direct and indirect cooling water (necessary pumpage included)[5]	1,443,000	85,000
Thermal regeneration of pickle liquor [6]	790,000	46,000
Ion exchange for chromium plating wastes [1]	216,000	13,000
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use)[1]	1,875,000	110,000
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,000
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling)[1]	790,000	47,000
Carbon columns (biological operation)[2]	2,666,000	157,000
80% Phosphorus removal[2]	86,000	5,000
Post aeration[2]	508,000	30,000
Conveyance to AWT plant trunk line[2]	265,000	16,000
Regional advanced biological AWT plant[2]	14,880,000	878,000
TOTAL	40,017,000	2,360,000

TABLE B-VI-B-5

NCED BIOLOGICAL TREATMENT(NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	
5,400,000	318,000	108,000	1,210,000	- 675,000	961,000
4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
1,200,000	71,000	24,000	73,000	-	168,000
1,443,000	85,000	28,000	14,000	-	127,000
790,000	46,000	16,000	217,000	-830,000	-551,000
216,000	13,000	4,000	70,000	-	87,000
1,875,000	110,000				110,000
5,400,000	319,000	108,000	140,000	- 67,000	500,000
790,000	47,000	16,000	1,465,000		1,528,000
2,666,000	157,000				157,000
86,000	5,000				5,000
508,000	30,000				30,000
265,000	16,000	5,000	3,000		24,000
14,880,000	878,000	298,000	996,000		2,172,000
40,017,000	2,360,000	697,000	4,556,000	-2,882,000	4,741,000

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B-VI-B-6

ANNUAL COST SUMMARY OF REGIONAL PHYSICAL-CHEMICAL TREATMENT (NDCP)

1972 DOLLARS

Process & Treatment	Capital Cost	Amortiz
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	31
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	26
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	7
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,441,000	8
Thermal regeneration of pickle liquor [6]	790,000	4
Ion exchange for chromium plating wastes [1]	216,000	1
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47
Carbon columns (biological operation) [2]	2,666,000	157
80% Phosphorus removal [2]	86,000	5
Post aeration [2]	508,000	30
Conveyance to AWT plant trunk line [2]	265,000	16
Regional physical-chemical AWT plant [2]	8,680,000	512
TOTAL	33,817,000	1,994

TABLE B-VI-B-6

CHEMICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
	5,400,000	318,000	108,000	1,210,000	-675,000	961,000
	4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
	1,200,000	71,000	24,000	73,000	-	168,000
	1,441,000	85,000	28,000	14,000	-	127,000
	790,000	46,000	16,000	217,000	-830,000	-551,000
	216,000	13,000	4,000	70,000	-	87,000
	1,875,000	110,000				110,000
1/ mill	5,400,000	319,000	108,000	140,000	- 67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
	2,666,000	157,000				157,000
	86,000	5,000				5,000
	508,000	30,000				30,000
	265,000	16,000	5,000	3,000		24,000
	8,680,000	512,000	174,000	996,000		1,682,000
	33,817,000	1,994,000	573,000	4,566,000	-2,882,000	4,251,000

B-VI-B-7

ANNUAL COST SUMMARY OF REGIONAL LAND TREATMENT (NDCP) VIA CURR

1972 DOLLARS

Process & Treatment	Capital Cost	Amortizat
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	71,
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,443,000	85,
Thermal regeneration of pickle liquor [6]	790,000	46,
Ion exchange for chromium plating wastes [1]	216,000	13,
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110,
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47,
Carbon columns (biological operation) [2]	2,666,000	157,
80% Phosphorus removal [2]	86,000	5,
Post aeration [2]	508,000	30,
Conveyance to AWT plant trunk line [2]	265,000	16,
Extra conveyance to land treatment site [2]		
Regional land AWT plant [2]	6,944,000	410,
TOTAL	32,081,000	1,892,

TABLE B-VI-B-7

D TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Annual Cost
	5,400,000	318,000	108,000	1,210,000	-675,000	961,000
	4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
	1,200,000	71,000	24,000	73,000	-	168,000
	1,441,000	85,000	28,000	14,000	-	127,000
	790,000	46,000	16,000	217,000	-830,000	-551,000
	216,000	13,000	4,000	70,000	-	87,000
	1,875,000	110,000				110,000
1/	5,400,000	319,000	108,000	140,000	-67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
	2,666,000	157,000				157,000
	86,000	5,000				5,000
	508,000	30,000				30,000
	265,000	16,000	5,000	3,000		3,000
				355,000		355,000
	6,944,000	410,000	139,000	371,000		920,000
	32,081,000	1,892,000	538,000	4,296,000	-2,882,000	3,884,000

B-VI-B-8

ANNUAL COST SUMMARY OF REGIONAL ADVANCED BIOLOGICAL TREATMENT (NDCP) VIA

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,000
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,000
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	71,000
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,443,000	85,000
Thermal regeneration of pickle liquor [6]	790,000	46,000
Ion exchange for chromium plating wastes [1]	216,000	13,000
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110,000
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,000
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47,000
Conveyance to AWT plant trunk line [2]	265,000	16,000
Regional advanced biological AWT plant [2]	14,880,000	878,000
TOTAL	36,757,000	2,168,000

TABLE B-VI-B-8

...OLOGICAL TREATMENT (NDCP) VIA PRESENT TREATMENT

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Annual Cost
	5,400,000	318,000	108,000	1,210,000	-675,000	961,000
	4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
	1,200,000	71,000	24,000	73,000	-	168,000
	1,443,000	85,000	28,000	14,000	-	127,000
	790,000	46,000	16,000	217,000	-830,000	-551,000
	216,000	13,000	4,000	70,000	-	87,000
	1,875,000	110,000				110,000
1/	5,400,000	319,000	108,000	140,000	- 67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
	265,000	16,000	5,000	3,000		24,000
	14,880,000	878,000	298,000	996,000		2,172,000
	36,757,000	2,168,000	697,000	4,566,000	-2,882,000	4,549,000

B-VI-B-9

ANNUAL COST SUMMARY OF REGIONAL PHYSICAL-CHEMICAL TREATMENT (NDCP) VIA

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,000
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,000
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	71,000
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,440,000	85,000
Thermal regeneration of pickle liquor [6]	790,000	46,000
Ion exchange for chromium plating wastes [1]	216,000	13,000
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110,000
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,000
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47,000
Conveyance to AWT plant trunk line [2]	265,000	16,000
Regional physical-chemical AWT plant [2]	8,680,000	512,000
TOTAL	30,557,000	1,802,000

TABLE B-VI-B-9

-CHEMICAL TREATMENT (NDCP) VIA PRESENT TREATMENT

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
	5,400,000	318,000	108,000	1,210,000	−675,000	961,000
	4,500,000	265,000	90,000	378,000	−1,310,000	−577,000
	1,200,000	71,000	24,000	73,000	−	168,000
	1,441,000	85,000	28,000	14,000	−	127,000
	790,000	46,000	16,000	217,000	−830,000	−551,000
	216,000	13,000	4,000	70,000	−	87,000
	1,875,000	110,000				110,000
mill $\frac{1}{}$	5,400,000	319,000	108,000	140,000	− 67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
	265,000	16,000	5,000	3,000		24,000
	8,680,000	512,000	174,000	996,000		1,682,000
	30,557,000	1,802,000	573,000	4,566,000	−2,882,000	4,059,000

B-VI-B-10

ANNUAL COST SUMMARY OF REGIONAL LAND TREATMENT (NDCP) VIA PRESENT

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
Plain sedimentation & recirculation of flue gas wash water [1]	5,400,000	318,000
Plain sedimentation & recirculation of scale breaking water [1]	4,500,000	265,000
High rate pressure filters for clarifer overflow (pumpage to cooling tower provided) [4]	1,200,000	71,000
Cooling towers for direct and indirect cooling water (necessary pumpage included) [5]	1,441,000	85,000
Thermal regeneration of pickle liquor [6]	790,000	46,000
Ion exchange for chromium plating wastes [1]	216,000	13,000
Pickle liquor & rinse neutralization with sludge lagooning (discontinued use) [1]	1,875,000	110,000
Palm oil recovery from flood lubricating wastewater from cold mill [1]	5,400,000	319,000
Treatment of sanitary, coke plant & pickle rinse water:		
Aerated lagoon (with sludge handling) [1]	790,000	47,000
Conveyance to AWT plant trunk line [2] Extra conveyance to land treatment site [2]	265,000	16,000
Regional land AWT plant [2]	6,944,000	410,000
TOTAL	28,821,000	1,700,000

TABLE B-VI-B-10

L LAND TREATMENT (NDCP) VIA PRESENT TREATMENT

1972 DOLLARS

	Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
/	5,400,000	318,000	108,000	1,210,000	-675,000	961,000
/	4,500,000	265,000	90,000	378,000	-1,310,000	-577,000
	1,200,000	71,000	24,000	73,000	-	168,000
sary	1,441,000	85,000	28,000	14,000	-	127,000
	790,000	46,000	16,000	217,000	-830,000	-551,000
	216,000	13,000	4,000	70,000	-	87,000
	1,875,000	110,000				110,000
d mill 1/	5,400,000	319,000	108,000	140,000	- 67,000	500,000
	790,000	47,000	16,000	1,465,000		1,528,000
	265,000	16,000	5,000	3,000		3,000
				355,000		355,000
	6,944,000	410,000	139,000	371,000		920,000
	28,821,000	1,700,000	538,000	4,296,000	-2,882,000	3,652,000

B-VI-B-11

ANNUAL COST SUMMARY OF MODULE TO ACHIEVE DES

PRESENT TREATMENT		TREATMENT FOR CURRENT STANDARDS

On Site

On Site

$3,715,000 $\longrightarrow$ $3,108,000 $\longrightarrow$

Adv. Biol.

Phys. Chem.

Adv. Biol.

Phys. Chem.

Land

Adv. Biol.

Phys. Chem.

Land
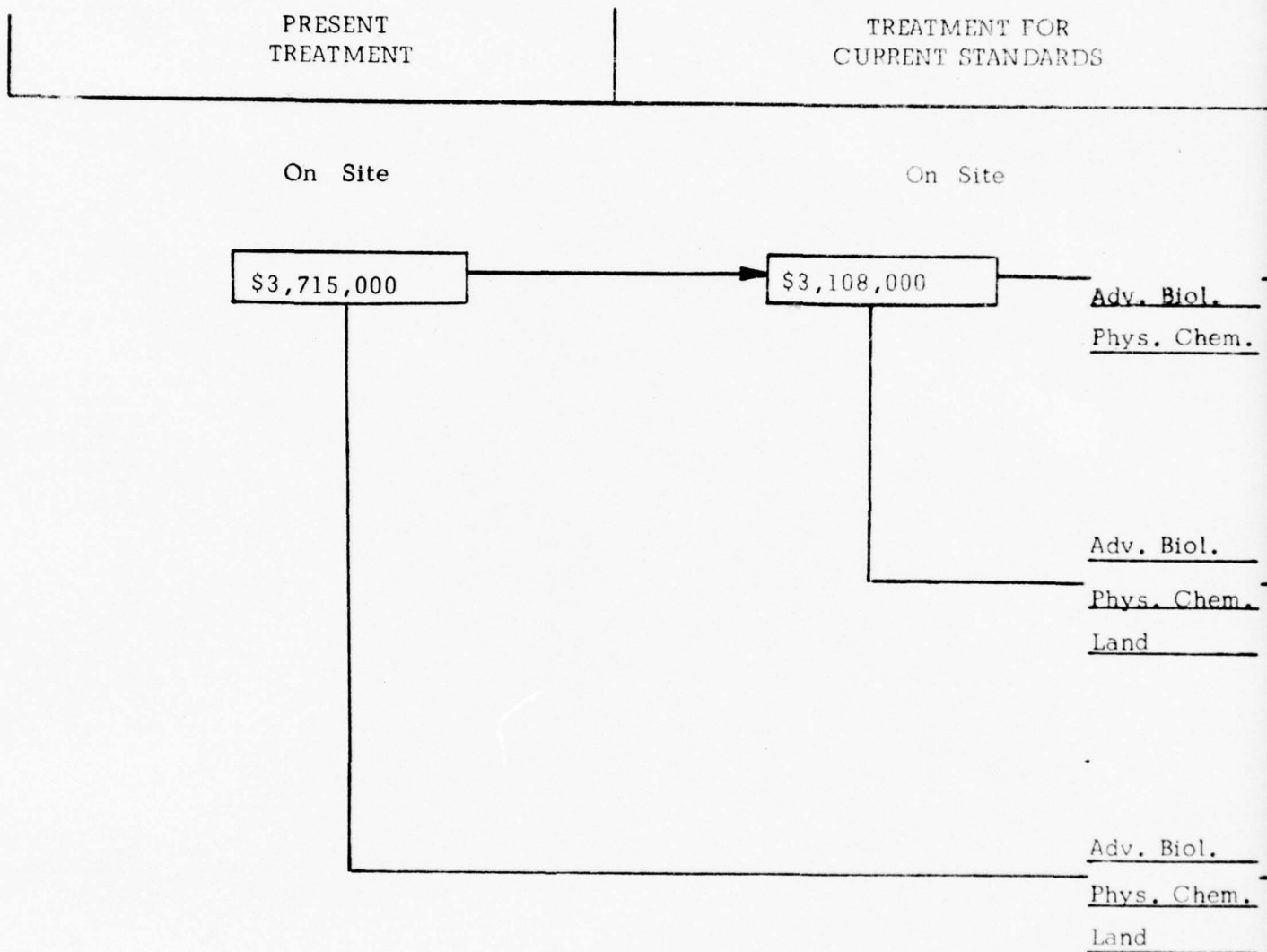
TABLE B-VI-B-11

SUMMARY OF MODULE TO ACHIEVE DESIRED TREATMENT - STEEL INDUSTRY

TREATMENT FOR CURRENT STANDARDS			TREATMENT FOR NDCP STANDARDS	

On Site

		COST OF PRIOR TREATMENT	On Site REGIONAL TREATMENT COST	TOTAL
$3,108,000	Adv. Biol.	$	$	$4,277,000
	Phys. Chem.			$4,278,000

Regional

	COST OF PRIOR TREATMENT	REGIONAL TREATMENT COST	TOTAL
Adv. Biol.	$2,569,000	$2,172,000	$4,741,000
Phys. Chem.	$2,569,000	$1,682,000	$4,251,000
Land	$2,569,000	$1,275,000	$3,844,000

Regional

	COST OF PRIOR TREATMENT	REGIONAL TREATMENT COST	TOTAL
Adv. Biol.	$2,377,000	$2,172,000	$4,549,000
Phys. Chem.	$2,377,000	$1,682,000	$4,059,000
Land	$2,377,000	$1,275,000	$3,652,000

B-VI-B-12

A replacement schedule has been designed which provides for 2% of capital cost per year. The column showing recovery credit was based on the value of the recovered usable material obtained for the particular treatment process by which it is listed. A bibliography has been provided to indicate the source material for the design and costs.

COST TO PETROLEUM INDUSTRY

The following tables B-VI-B-12 through 21 contain detailed estimates of costs for petroleum industry wastewater treatment varying from present practices through NDCP standards. The alternatives presented in these tables are of the same format described for the steel industry. Table B-VI-B-22 is a summary of the total annual costs per module for each path listed in the tables above.

Cooling tower costs are calculated on the basis of the quantity of heat dissipated from data provided in Reference 5.

A replacement schedule has been designed which provides for 2% of capital cost per year. Oil recovered in separators and slop oil treatment is a definite benefit, and should be credited to the cost of the treatment process which provides for this recovery. However, due to the variety of operations at each refinery and the sequence of the sub process mix, it is impossible to quantify the recovery credit for oil as a generally constant figure. This will have to be done at each refinery, and the credit may vary not only on operations, but also on the composition of the raw materials. A bibliography is included to indicate the sources of data for the designs and costs.

NON-CRITICAL INDUSTRY

The characteristics of wastewater vary markedly from industry to industry within the "non-critical" category. For this reason it is not possible to assign a general or typical treatment technology to the non-critical industries. For an analogous reason it is not possible to compute module unit costs for the non-critical industries.

An average cost per unit of flow treated can be assigned by using the weighted average of the steel and petroleum industry cost to treat unit flow. Therefore, the total annual cost to non-critical industry can be determined. This cost is presented in Appendix D.

ANNUAL COST SUMMARY OF PRESENT TREATM
FACILITIES AT TYPICAL REFINERY IN C-SELM A
1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator	158,000	9,000
Sour Water Stripping	231,000	14,000
Slop Oil Treatment	171,000	10,000
Carbon Column Polishing	3,763,000	222,000
Activated Sludge	710,000	42,000
Sludge Thichening Vacuum Filtration & Incineration	422,000	25,000
Chlorination	304,000	18,000
TOTALS	5,759,000	340,000

*Some credit due to oil recovery (Difficult to quantify)

TABLE B-VI-B-12

AL COST SUMMARY OF PRESENT TREATMENT
ITIES AT TYPICAL REFINERY IN C-SELM AREA
1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O & M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
231,000	14,000	5,000	23,000	–	42,000
171,000	10,000	3,000	17,000	*	30,000
3,763,000	222,000	75,000	370,000		667,000
710,000	42,000	14,000	147,000	–	203,000
422,000	25,000	8,000	80,000	–	113,000
304,000	18,000	6,000	98,000		72,000
5,759,000	340,000	114,000	718,000		1,172,000

ANNUAL COST SUMMARY OF TREATMENT FACILITIES TO MEET CURRENT STANDARDS IN

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator 7/	158,000	9,000
Slop Oil Treatment 7/	171,000	10,000
High Rate Pressured Filters 4/	1,700,000	100,000
Cooling Towers & Required Pumpage 5/	1,090,000	65,000
Sour Water Stripping 7/	231,000	14,000
Activated Sludge & Sludge Handling 2,7/	1,132,000	67,000
Carbon Column Polishing 2/	3,763,000	222,000
Chlorination 2/	304,000	18,000
Post Aeration 2/	272,000	16,000
TOTAL	8,821,000	521,000

S TO MEET CURRENT STANDARDS IN C-SELM AREA

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
1,700,000	100,000	34,000	105,000	-	239,000
1,090,000	65,000	22,000	11,000	-	98,000
231,000	14,000	5,000	23,000	-	42,000
1,132,000	67,000	2,000	33,000	-	102,000
3,763,000	222,000	75,000	141,000	-	438,000
304,000	18,000	6,000	7,000	-	31,000
272,000	16,000	5,000	38,000	-	59,000
8,821,000	521,000	155,000	408,000	-	1,084,000

B-VI-B-15

ANNUAL COST SUMMARY OF ON-SITE ADVANCED BIOLOGICAL TREATMENT (N

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator [7]	158,000	9,00
Slop Oil Treatment [7]	171,000	10,00
Final Filters [2,4]	1,700,000	100,00
Cooling Towers & Required Pumpage [5]	1,090,000	65,00
Sour Water Stripping [7]	231,000	14,00
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,00
Carbon Column Polishing [2]	3,763,000	222,00
Chlorination [2]	304,000	18,00
Post Aeration [2]	272,000	16,00
98% Phosphorus Removal [2]	1,500,000	89,00
Nitrification-Denitrification [2]	2,640,000	156,00
TOTAL	12,961,000	766,00

TABLE B-VI-B-14

NCED BIOLOGICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
1,700,000	100,000	12,000	41,000	-	153,000
1,090,000	65,000	22,000	11,000	-	98,000
231,000	14,000	5,000	23,000	-	42,000
1,132,000	67,000	2,000	33,000	-	102,000
3,763,000	222,000	75,000	141,000	-	438,000
304,000	18,000	6,000	7,000	-	31,000
272,000	16,000	5,000	38,000	-	59,000
1,500,000	89,000	30,000	161,000	-	280,000
2,640,000	156,000	53,000	95,000	-	304,000
12,961,000	766,000	216,000	600,000	-	1,582,000

B-VI-B-16

ANNUAL COST SUMMARY OF ON-SITE PHYSICAL-CHEMICAL TREATMENT (NDCP)

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator [7]	158,000	9,000
Slop Oil Treatment [7]	171,000	10,000
Final Filters [2,4]	1,700,000	100,000
Cooling Towers & Required Pumpage [5]	1,090,000	65,000
Sour Water Stripping [7]	231,000	14,000
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,000
Carbon Column Polishing [2]	3,763,000	222,000
Chlorination [2]	304,000	18,000
Post Aeration [2]	272,000	16,000
98% Phosphorus Removal [2]	1,500,000	89,000
Nitrogen Removal with Clinoptilolite [2]	1,120,000	66,000
TOTAL	11,441,000	676,000

TABLE B-VI-B-15

L-CHEMICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
1,700,000	100,000	12,000	41,000	–	153,000
1,090,000	65,000	22,000	11,000	–	98,000
231,000	14,000	5,000	23,000	–	42,000
1,132,000	67,000	2,000	33,000	–	102,000
3,763,000	222,000	75,000	141,000	–	438,000
304,000	18,000	6,000	7,000	–	31,000
272,000	16,000	5,000	38,000	–	59,000
1,500,000	89,000	30,000	161,000	–	280,000
1,120,000	66,000	22,000	190,000	–	278,000
11,441,000	676,000	185,000	695,000	–	1,556,000

B-VI-B-17

ANNUAL COST SUMMARY OF REGIONAL ADVANCED BIOLOGICAL TREAT

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator [7]	158,000	9,000
Slop Oil Treatment [7]	171,000	10,000
High Rate Pressured Filters [4] (Discontinued Use)	1,700,000	100,000
Cooling Towers & Required Pumpage [5]	1,090,000	65,000
Sour Water Stripping [7]	231,000	14,000
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,000
Carbon Column Polishing [2] (Discontinued Use)	3,763,000	222,000
Chlorination [2] (Discontinued Use)	304,000	18,000
Post Aeration [2] (Discontinued Use)	272,000	16,000
Conveyance to AWT plant trunk line [2]	174,000	10,000
Regional advanced biological AWT plant [2]	4,800,000	283,000
TOTAL	13,795,000	814,000

TABLE B-VI-B-16

REGIONAL ADVANCED BIOLOGICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
1,700,000	100,000	–	–	–	100,000
1,090,000	65,000	22,000	11,000	–	98,000
231,000	14,000	5,000	23,000	–	42,000
1,132,000	67,000	2,000	33,000	–	102,000
3,763,000	222,000	–	–	–	222,000
304,000	18,000	–	.	–	18,000
272,000	16,000	–	–	–	16,000
174,000	10,000	4,000	1,000	–	15,000
4,800,000	283,000	96,000	321,000	–	700,000
13,795,000	814,000	135,000	439,000	–	1,388,000

B-VI-B-18

ANNUAL COST SUMMARY OF REGIONAL PHYSICAL-CHEMICAL TREATMENT (ND

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator [7]	158,000	9,000
Slop Oil Treatment [7]	171,000	10,000
High Rate Pressured Filters [4] (Discontinued Use)	1,700,000	100,000
Cooling Towers & Required Pumpage [5]	1,090,000	65,000
Sour Water Stripping [7]	231,000	14,000
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,000
Carbon Column Polishing [2] (Discontinued Use)	3,763,000	222,000
Chlorination [2] (Discontinued Use)	304,000	18,000
Post Aeration [2] (Discontinued Use)	272,000	16,000
Conveyance to AWT plant trunk line [2]	174,000	10,000
Regional Physical-Chemical AWT Plant [2]	2,800,000	165,000
TOTAL	11,795,000	696,000

TABLE B-VI-B-17

L PHYSICAL-CHEMICAL TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
1,700,000	100,000	−	−	−	100,000
1,090,000	65,000	22,000	11,000	−	98,000
231,000	14,000	5,000	23,000	−	42,000
1,132,000	67,000	2,000	33,000	−	102,000
3,763,000	222,000	−	−	−	222,000
304,000	18,000	−	−	−	18,000
272,000	16,000	−	−	−	16,000
174,000	10,000	4,000	1,000	−	15,000
2,800,000	165,000	56,000	321,000	−	542,000
11,795,000	696,000	95,000	439,000	−	1,230,000

ANNUAL COST SUMMARY OF REGIONAL LAND TREATMENT (NDCP) VIA CURREN

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization	
API Separator [7]	158,000	9,000	
Slop Oil Treatment [7]	171,000	10,000	
High Rate Pressured Filters [4] (Discontinued Use)	1,700,000	100,000	
Cooling Towers & Required Pumpage [5]	1,090,000	65,000	
Sour Water Stripping [7]	231,000	14,000	
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,000	
Carbon Column Polishing [2] (Discontinued Use)	3,763,000	222,000	
Chlorination [2] (Discontinued Use)	304,000	18,000	
Post Aeration [2] (Discontinued Use)	272,000	16,000	
Conveyance to AWT plant trunk line [2]	174,000	10,000	
Extra conveyance to land treatment site [2]	-	-	
Regional Land AWT Plant [2]	2,240,000	**132,000**	
TOTAL	11,235,000	663,000	

TABLE B-VI-B-18

LAND TREATMENT (NDCP) VIA CURRENT STANDARDS

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
1,700,000	100,000	-	.	-	100,000
1,090,000	65,000	22,000	11,000	-	98,000
231,000	14,000	5,000	23,000	-	42,000
1,132,000	67,000	2,000	33,000	-	102,000
3,763,000	222,000	-	.	-	222,000
304,000	18,000	-	-	-	18,000
272,000	16,000	-	.	-	16,000
174,000	10,000	4,000	1,000	-	15,000
-	-	-	115,000	-	115,000
2,240,000	**132,000**	45,000	120,000	-	297,000
11,235,000	663,000	84,000	353,000	-	1,100,000

B-VI-B-20

ANNUAL COST SUMMARY OF REGIONAL ADVANCED BIOLOGICAL TREATMENT (

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator [7]	158,000	9,000
Slop Oil Treatment [7]	171,000	10,000
Conveyance to AWT plant trunk line [2]	174,000	10,000
Cooling Towers & Required Pumpage [5]	1,090,000	65,000
Sour Water Stripping [7]	231,000	14,000
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,000
Carbon Column Polishing [2] (Discontinued Use)	3,763,000	222,000
Chlorination [2] (Discontinued Use)	304,000	18,000
Regional Advanced Biological AWT Plant [2]	4,800,000	283,000
TOTAL	11,823,000	698,000

TABLE B-VI-B-19

ADVANCED BIOLOGICAL TREATMENT (NDCP) VIA PRESENT TREATMENT

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
174,000	10,000	4,000	1,000	-	15,000
1,090,000	65,000	22,000	11,000	-	98,000
231,000	14,000	5,000	23,000	-	42,000
1,132,000	67,000	2,000	33,000	-	102,000
3,763,000	222,000	-	-	-	222,000
304,000	18,000	-	-	-	18,000
4,800,000	283,000	96,000	321,000	-	700,000
11,823,000	698,000	135,000	439,000	-	1,272,000

B-VI-B-21

ANNUAL COST SUMMARY OF REGIONAL PHYSICAL-CHEMICAL TREATMENT

1972 DOLLARS

Process & Treatment	Capital Cost	Amortization
API Separator [7]	158,000	9,000
Slop Oil Treatment [7]	171,000	10,000
Conveyance to AWT plant trunk line [2]	174,000	10,000
Cooling Towers & Required Pumpage [5]	1,090,000	65,000
Sour Water Stripping [7]	231,000	14,000
Activated Sludge & Sludge Handling [2,7]	1,132,000	67,000
Carbon Column Polishing [2] (Discontinued Use)	3,763,000	222,000
Chlorination [2] (Discontinued Use)	304,000	18,000
Regional Physical-Chemical AWT Plant [2]	2,800,000	165,000
TOTAL	9,823,000	580,000

TABLE B-VI-B-20

SICAL-CHEMICAL TREATMENT (NDCP) VIA PRESENT TREATMENT

1972 DOLLARS

al Cost	Amortization	Replacement Cost	O&M	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
174,000	10,000	4,000	1,000	-	15,000
090,000	65,000	22,000	11,000	-	98,000
231,000	14,000	5,000	23,000	-	42,000
132,000	67,000	2,000	33,000	-	102,000
763,000	222,000	-	-	-	222,000
304,000	18,000	-	-	-	18,000
800,000	165,000	56,000	321,000	-	542,000
823,000	580,000	95,000	439,000	-	1,114,000

-VI-B-22

ANNUAL COST SUMMARY OF REGIONAL LAND TREATMENT (NE

1972 DOLLARS

Process & Treatment	Capital Cost	Amortiza
API Separator[7]	158,000	9
Slop Oil Treatment[7]	171,000	10
Regional Land AWT Plant[2]	2,240,000	132
Cooling Towers & Required Pumpage[5]	1,090,000	65
Sour Water Stripping[7]	231,000	14
Activated Sludge & Sludge Handling[2,7]	1,132,000	67
Carbon Column Polishing (Discontinued Use)[2]	3,763,000	222
Chlorination (Discontinued Use)[2]	304,000	18
Conveyance to AWT plant trunk line[2]	174,000	10
Extra conveyance to land treatment site[2]	-	
TOTAL	9,263,000	547

TABLE B-VI-B-21

REGIONAL LAND TREATMENT (NDCP) VIA PRESENT TREATMENT

1972 DOLLARS

Capital Cost	Amortization	Replacement Cost	OSM	Credit	Total Annual Cost
158,000	9,000	3,000	33,000	*	45,000
171,000	10,000	3,000	17,000	*	30,000
2,240,000	132,000	45,000	120,000	-	297,000
1,090,000	65,000	22,000	11,000	-	98,000
231,000	14,000	5,000	23,000	-	42,000
1,132,000	67,000	2,000	33,000	-	102,000
3,763,000	222,000	-	-	-	222,000
304,000	18,000	-	-	-	18,000
174,000	10,000	4,000	1,000	-	15,000
-	-	-	115,000	-	115,000
9,263,000	547,000	84,000	353,000	-	984,000

B-VI-B-23

ANNUAL COST SUMMARY OF MODULE TO ACHIEVE DESIRED

PRESENT TREATMENT	TREATMENT FOR CURRENT STANDARDS

On Site

On Site

$1,172,000 → $1,084,000 → Adv. Biol.

Phys. Chem.

Adv. Biol.

Phys. Chem.
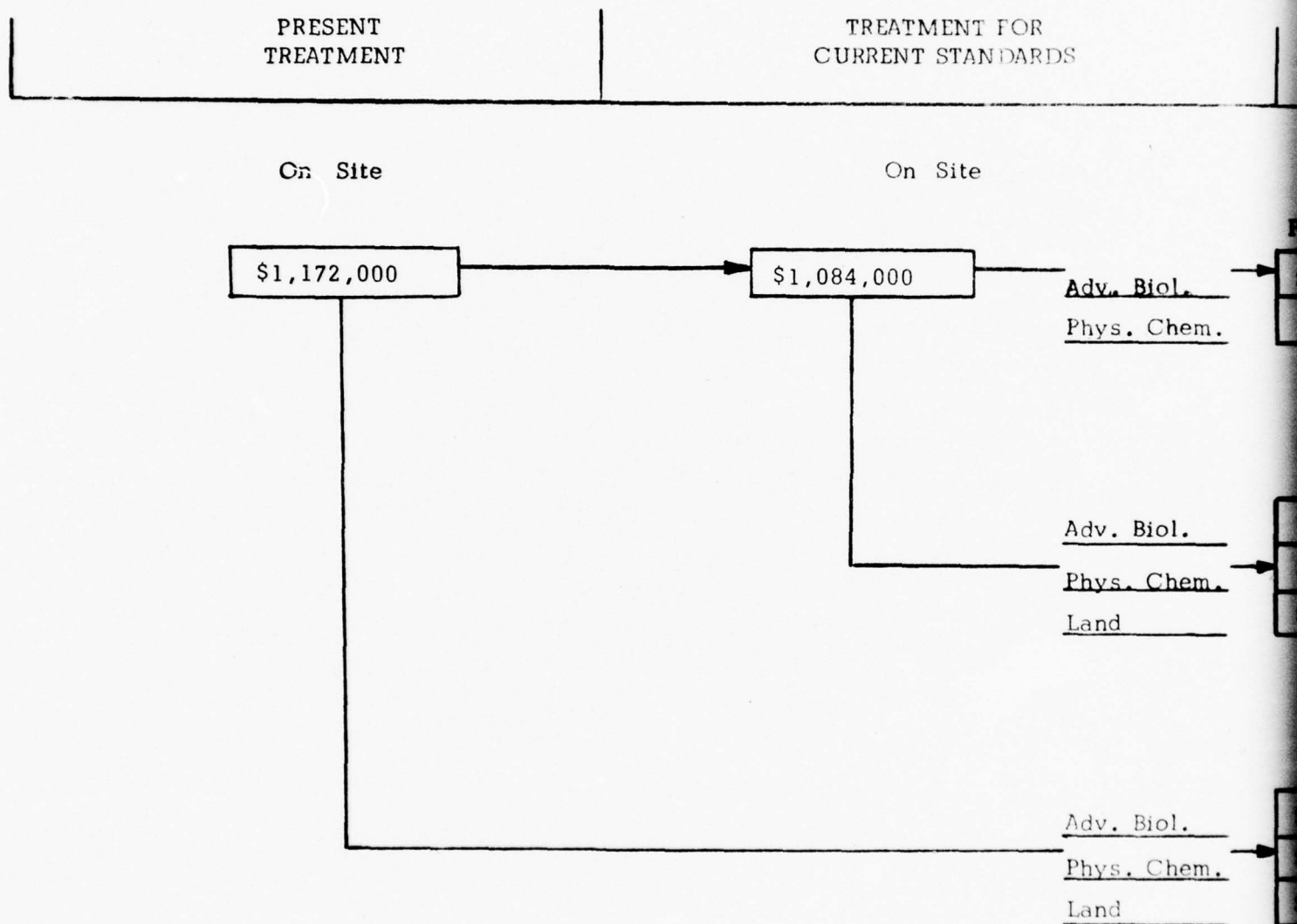
Land

Adv. Biol.

Phys. Chem.

Land

TABLE B-VI-B-22

SUMMARY OF MODULE TO ACHIEVE DESIRED TREATMENT - PETROLEUM INDUSTRY

TREATMENT FOR CURRENT STANDARDS		TREATMENT FOR NDCP STANDARDS		

On Site

		COST OF PRIOR TREATMENT	On Site REGIONAL TREATMENT COST	TOTAL
$1,084,000	Adv. Biol.	$	$	$1,582,000
	Phys. Chem.			$1,556,000

Regional

	COST OF PRIOR TREATMENT	REGIONAL TREATMENT COST	TOTAL
Adv. Biol.	$688,000	$700,000	$1,388,000
Phys. Chem.	$688,000	$542,000	$1,230,000
Land	$688,000	$412,000	$1,100,000

Regional

	COST OF PRIOR TREATMENT	REGIONAL TREATMENT COST	TOTAL
Adv. Biol.	$572,000	$700,000	$1,272,000
Phys. Chem.	$572,000	$542,000	$1,114,000
Land	$572,000	$412,000	$ 984,000

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VI. COMPONENT BASIS OF COST

C. SLUDGE MANAGEMENT SYSTEMS

INTRODUCTION

In developing the component basis of cost for the C-SELM sludge management systems, only those aspects covered in the Sludge Management Systems sub-sections of Appendix B, Section IV-C are considered. As this sub-section dealt in detail with two components from the overall sludge management techniques used, only the basis of cost for these two components - sludge transportation and land application systems - is given in this section. The basis of cost for the sludge handling and stabilization components of the treatment plant systems is found in Appendix B, Section VI-A.

Sludge Transportation

Analyses of costs associated with sludge transportation systems indicate that transportation cost varies with the solids content of the sludge. A preliminary cost analysis, comparing each mode of sludge transporation including pipeline, truck, barge, and railroad, indicates that a pipeline system is the most economical means of transportation when the solids content of the sludge transported is maintained at the 6% level for biological sludges and 10% for physical-chemical sludges. This analysis is based on the assumption that a railroad or waterway exists between the transfer station and the land application site.

In the final determination of the biological sludge transportation costs for this study it is assumed that sludge thickening, combined with barge or railroad systems of transportation, could produce unit costs comparable to those for the pipeline system. It is also assumed that adjustment of the solids content of the physical-chemical sludge to a easy loading-and-unloading concentration for the railroad or barge system could produce unit costs comparable to those for the pipeline system. The transportation of physical-chemical ash by a truck system is not considered because of the handling problems associated with this system and because of high operation and maintenance costs. While the costs for a pipeline transportation system are used as the sludge transportation costs for this study, they are not necessarily any less than the costs for the most economic version of either of the alternative rail or barge transportation systems.

The modular costs associated with the main sludge transportation systems apply to the tributary sludge collection systems as well.

The unit costs for a pipeline transportation system at various flows are developed using the following basic assumptions.

1. Pipe sizes and the horsepower required at pumping stations for a given flow of sludge are determined using the design bases described in Appendix B, Section IV-C.

2. The average cost of installed pipeline equals $2 per inch of diameter/linear foot of pipe.

3. The cost of the pumping stations is taken from the unit costs of pumping stations developed for waste-water conveyance, Figure B-VI-C-1, and corrected using a factor of 1.32 to compensate for the increase in power and in physical size of the pumps and motors.

4. Labor costs equal 1.8% of the construction cost per year.

5. Energy costs are based on a unit cost of $0.01/KWH and 24 hours per day operation.

6. Maintenance and supplies equal 0.6% of the construction cost.

7. Replacement costs for a pumping station and associated pipeline are computed using the following schedule

REPLACEMENT SCHEDULE

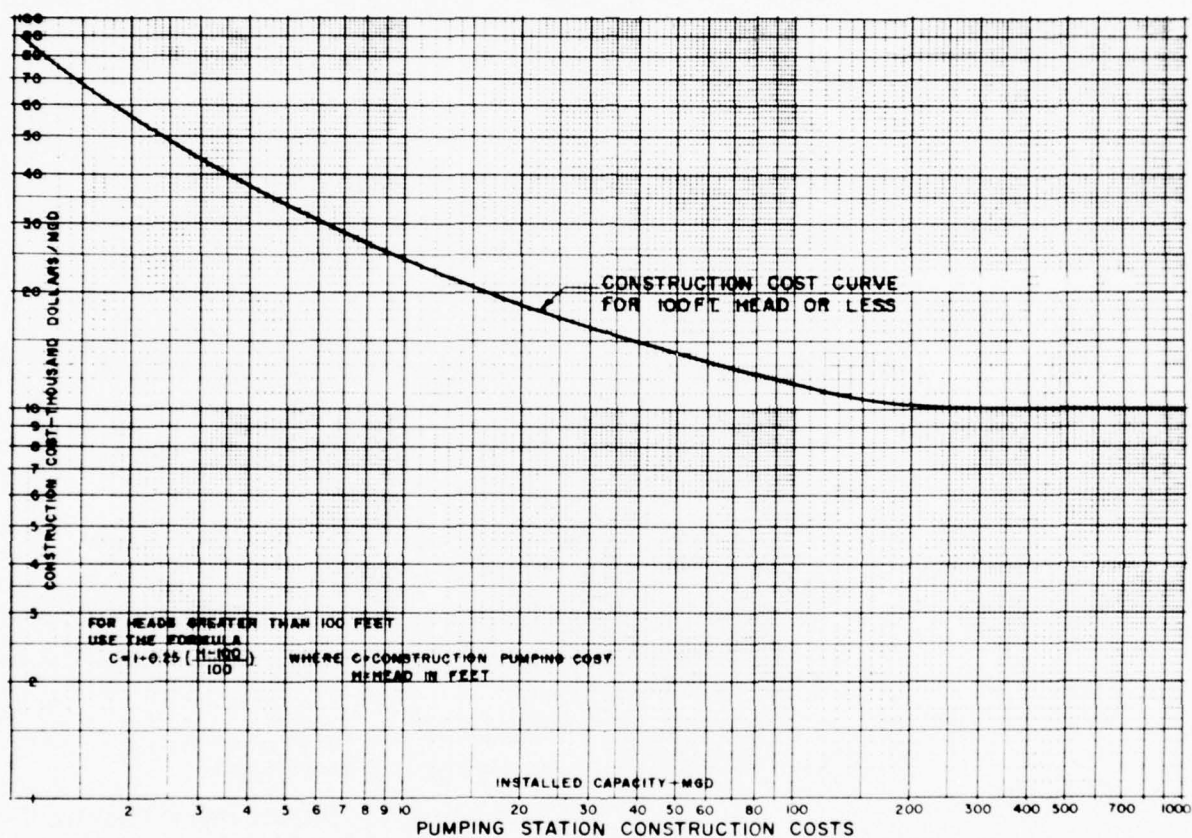| Components	% of Total Cost	Replacement Required in Years
Land & Structures	20%	No Replacement in 50 Yrs.
Structures and pipeline	60%	Every 25 Yrs.
Mechanical	10%	Every 10 Yrs.
Other	5%	Every 10 Yrs.
Other	5%	No Replacement

Figure B-VI-C-1

PUMPING STATION CONSTRUCTION COSTS

B-VI-C-3

The results of this cost analysis are presented in graphic form as Figure B-VI-C-2.

LAND APPLICATION SYSTEMS

The costs for the two land application systems, agricultural utilization and land reclamation, are analyzed for this study. The costs for the sludge distribution system, for land clearing, and for the construction of sludge storage lagoons are included in each system cost. The unit costs for land application systems are developed using the following basic assumptions.

1. The system designs are based on the design criteria described in Appendix B, Section IV-C.

2. The same methodology for the pipeline transportation system is used here for the computation of pumping station and pipeline costs.

3. The costs of fittings are based on responses from manufacturers and contractors for each particular type and size required. No general rule is used.

4. The cost of a tractor plus plow is assumed to be $32,000.

5. The cost of a physical-chemical sludge applicator and a tractor plus sprinkler is assumed to be $25,000.

6. Land clearing costs for agricultural utilization and land reclamation are $100/ac and $500/ac, respectively.

7. The replacement schedule for the agricultural utilization application system is as follows:

Components	% of Total Cost	Replacement Required in Years
Pump Station	1.4%	10 years
Distribution System	1.3%	10 years
Tractor	8.8%	10 years
Pump Station	5.8%	25 years
Distribution System	62.0%	25 years

8. The land reclamation application system is designed so that once the desired quantity of sludge is applied to a 1400 acre unit, the system is abandoned and a new one is utlized on adjacent lands. Thus in a strict
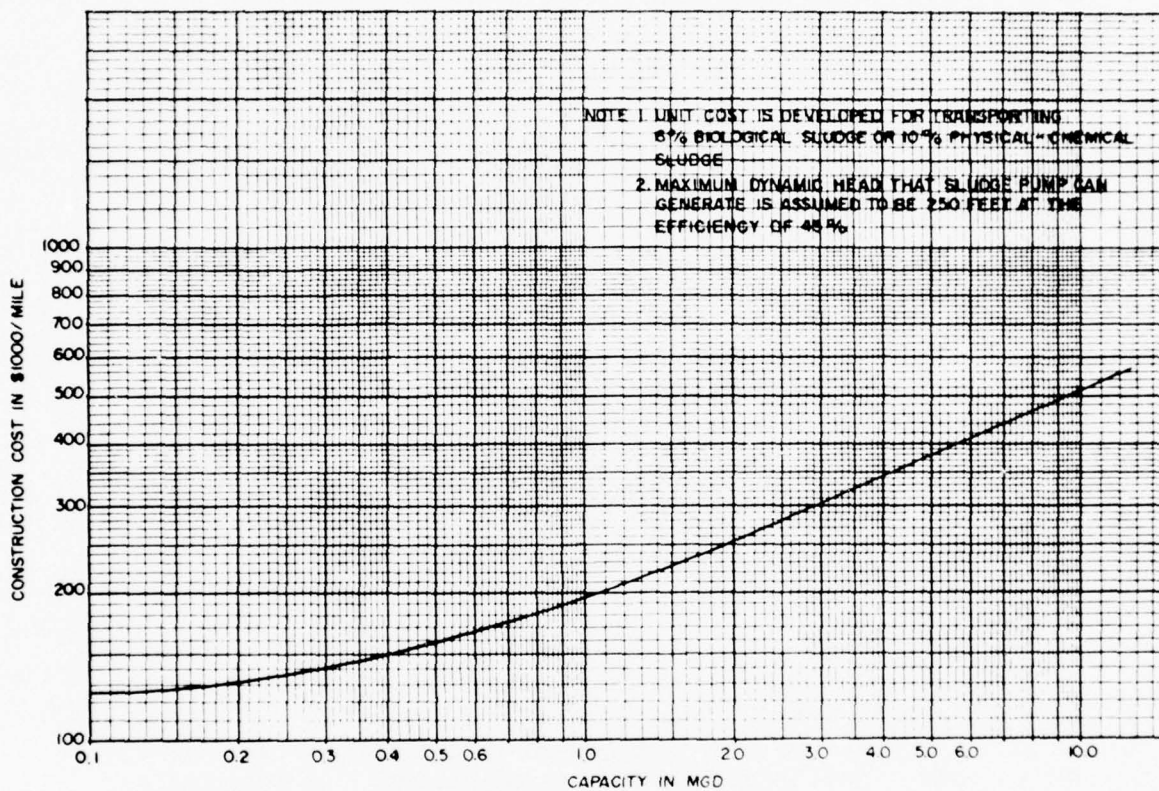
B-VI-C-4

NOTE 1. UNIT COST IS DEVELOPED FOR TRANSPORTING
6% BIOLOGICAL SLUDGE OR 10% PHYSICAL-CHEMICAL
SLUDGE

2. MAXIMUM DYNAMIC HEAD THAT SLUDGE PUMP CAN
GENERATE IS ASSUMED TO BE 250 FEET AT THE
EFFICIENCY OF 45%

Figure B-VI-C-2

UNIT COST OF PIPELINE SYSTEM CONSTRUCTION

sense, there is no replacement schedule for
this sludge application system. However,
the construction of all the sludge applica-
tion units during a five or ten year period
is not feasible since certain units would not
be utilized for 20 to 40 years after their
construction. Thus a construction schedule
is substituted for the replacement schedule.
The initial construction is designed to accom-
modate one-fourth of the total sludge application
requirements. At 10 year intervals, three more
application system construction projects are
programmed with each providing one-fourth
of the total system requirements.

Typical cost analyses for the land application of sludge are
given in Table B-VI-C-1 for agricultural utilization and in Table B-VI-
C-2 for land reclamation. These costs are based upon the layouts of
Figure B-IV-C-3 and Figure B-IV-C-6 including an allowance of
$1,000,000 per unit for distribution costs from the end of the main
sludge transportation system to each unit. This allowance is re-
quired because of the commonly scattered locations of application
areas within a general region, requiring additional pumping stations
and pipelines to convey the sludge from the main supply point in
that region to each new distribution unit being developed.

LAND COSTS

The use of land for agricultural sludge utilization is secured
by payments to the owners, equivalent to the current market price
of the land, but divided into a lump-sum initial payment and an
annual payment. The initial payment is ten percent of the land's
current market value to compensate the owner for potential crop losses
during the sludge management system installation and for the neces-
sary changes in farming practice occasioned by the constraints of the
sludge management system. The annual payment is equal to the
amortized value of ninety percent of the land's current market value
at the given interest rate over a fifty-year period. The annual pay-
ment is to compensate the present or future owner for his inability to real-
ize gain in land value for alternative land uses for a fifty-year time period.
The current market values of the farmland near the C-SELM study area are
summarized in Data Annex B, Section VII-B.

The use of land for land-reclamation sludge utilization is assumed
to be secured without payment to owners of strip-mined land in exchange
for the increased land value which results from the reclamation.

B-VI-C-6

Table B-VI-C-1

LAND APPLICATION UNIT COST FOR AGRICULTURAL UTILIZATION
(REPLACEMENT COSTS NOT INCLUDED)

Items	Land Treat. & Conv.Bio.Sludge	Advanced Biol.Sludge	Physical-Chem.Sludge
DESIGN CONDITIONS			
Area of Appl. unit (Ac)	5,151	5,151	5,151
Rate of Sludge Appl. (Dry Ton/Ac-Yr)	13.5	28.8	1.73
Design of Flow (MGD)	4.8	10.2	0.6
CONSTRUCTION COST ANALYSIS			
Pumping Station	226,000	355,000	125,000
Distribution System	2,000,000	2,460,000	1,358,000
Sludge Applicators (10)	320,000	320,000	320,000
Land Clearing	515,000	515,000	515,000
Total Cost less Cont.	$3,061,000	$3,650,000	$2,318,000
Contingencies & Eng.	1,108,000	1,387,000	881,000
Total Const. Cost	$4,169,000	$5,037,000	$3,199,000
O & M COST			
Labor	179,000	179,000	179,000
Power	5,500	11,000	700
Fuel	18,000	18,000	9,000
Total	$ 202,500	$ 208,000	$ 188,700

Table B-VI-C-2

LAND APPLICATION UNIT COST FOR LAND RECLAMATION
(LAND LEVELING COSTS NOT INCLUDED)
(REPLACEMENT COSTS NOT INCLUDED)

Items	Land Treat. Sludge	Adv. Bio. Sludge
DESIGN CONDITIONS		
Area of Application (Ac)	1,400	1,400
Rate of Application (Dry Ton/Ac)	100	213
Design Flow (MGD)	20	43
CONSTRUCTION COST		
Distribution System	1,800,000	2,170,000
Tractors & Sprinklers (7)	175,000	175,000
Land Clearing	700,000	700,000
Total	$2,755,000	$3,045,000
Eng. & Contingencies	1,047,000	1,157,000
Total Cost	$3,802,000	$4,202,000
& M COST		
Labor	179,000	179,000
Power	23,000	50,000
Fuel	9,000	9,000
Total $/Yr.	$ 211,000	$ 238,000

VI.   COMPONENT BASIS OF COST

D.   STORMWATER MANAGEMENT SYSTEMS

URBAN STORMWATER MANAGEMENT COSTS

The costs connected with the management of urban C-SELM storm-water are taken from the Summary of Technical Reports by the Flood Control Coordinating Committee dated August, 1972, for the Chicago Underflow Plan. These costs include surface collection and drop shafts, conveyance tunnels, storage reservoir facilities, pumping stations and discharge conduits. No additional costs are contemplated for this component of stormwater management and costs proposed by the above report are incorporated into the total cost of the C-SELM Wastewater Management System.

SUBURBAN STORMWATER MANAGEMENT COSTS

Suburban stormwater management costs consist mainly of expenditures necessary to create storage for stormwater and combined sewer flows. This storage is provided via shallow-pit units or mined storage units.

Shallow-Pit Storage Units

Cost components for the shallow-pit storage system includes costs of land, excavation and earthwork, landscaping, underdrainage system, aeration system, and grit removal system. Pumping station cost is included with the cost of the conveyance system for conveying stored flows to the treatment facility. A number of typical shallow pit storage units were costed to determine if significant economies are obtainable by varying the size, shape and depth of the unit. Land costs were varied for each county to reflect the prevailing average cost for open land. The unit cost per acre-foot of surface storage decreased slightly for sites in excess of 600 acre-feet of live storage and increased considerably for storage capacity of less than this amount. An average cost of $3,000 per acre-foot was selected as representative of this type of storage. Following is the breakdown of the above sum: all values in dollars per acre-foot of storage.

Land	$ 280.00
Excavation and Earthwork	1,300.00
Landscaping	50.00
Underdrainage	50.00
Aeration	1,000.00
Grit Removal	320.00
TOTAL	$3,000.00/ac.ft.

Mined Storage Units

Mined storage units are required in high density developed suburbs and towns, other than Chicago, which have combined sewer systems. There are 18 such mined storage units included in the C-SELM suburban stormwater management system.

The unit costs of hard-rock underground mining have a significant bearing on the costs of managing stormwater in suburban areas. Such mines can frequently be constructed in strategic locations without disruption of present surface facilities to provide the storage volume required to manage the storm runoff from combined and storm sewers. Estimates of cost of such mining range from $3 to $15 per cubic yard of space developed, depending upon a number of significant variables. Assuming a room-and-pillar mining system in a strong, massive, self-supporting formation, the following factors are among the most important in determining the actual costs of construction.

(a) Whether construction trades or mining workers unions are involved in the work.

(b) The total quantity of rock to be removed from one location. A large volume justifies the use of more and better materials handling equipment.

(c) The method of disposal of the mined rock. Having a systematic and efficient means for disposal can greatly reduce construction costs.

Unit costs for hard-rock underground mining of limestone by room-and-pillar methods seldom exceed $1.50 per ton for large commercial operations. This unit cost corresponds to about $3.00 per cubic yard of

rock removed. Many large mines achieve lower costs, on the order of $1 per ton or $2 per cubic yard. Such costs are achieved by mines which have operated continuously over several decades at volumes of at least 1,000 tons per day. It is not expected that such low unit costs would be achieved in mining dolomite for storage of stormwater as the duration of the mining would probably not exceed 10 years.

Aeration facilities in the mined storages would be constructed throughout the mined space. Compressed air bubbler pipes could be used for aeration, as could floating aerators.

Grit removal would be performed at intervals of several years using front-end loaders and trucks. Only heavier solids would deposit in the storage facilities as the passage down the vertical shafts at high speeds (50 feet per second) would tend to produce mostly small and easily suspended solids. The storage would probably accumulate a few feet of grit over a period of 10 years or so. This would be excavated during winter season when there would be little likelihood of storm runoff.

Storage costs include the cost of the access shaft, rock excavations, aeration facility, aquifer protection and other costs such as grit removal. On a per acre-foot basis storage costs vary from $23,500 ac/ft for Galena formations to $28,000 ft for Niagaran formations. Following is a breakdown of the above sums. All values are in dollars per acre-foot of storage.

Item	Galena		Niagaran
Access Shaft	1,000.00		1,500.00
Rock Excavation @$10.00 cy	16,130.00	@$12.00 cy	17,743.00
Aeration	4,000.00		4,000.00
Aquifer Protection	1,870.00		4,000.00
Grit Removal	500.00		757.00
TOTAL	23,500.00		28,000.00

The curve depicting cost of mined storage is enclosed as Figure B-VI-D-1. The curve depicting cost of drop shaft is enclosed as Figure B-VI-D-2.
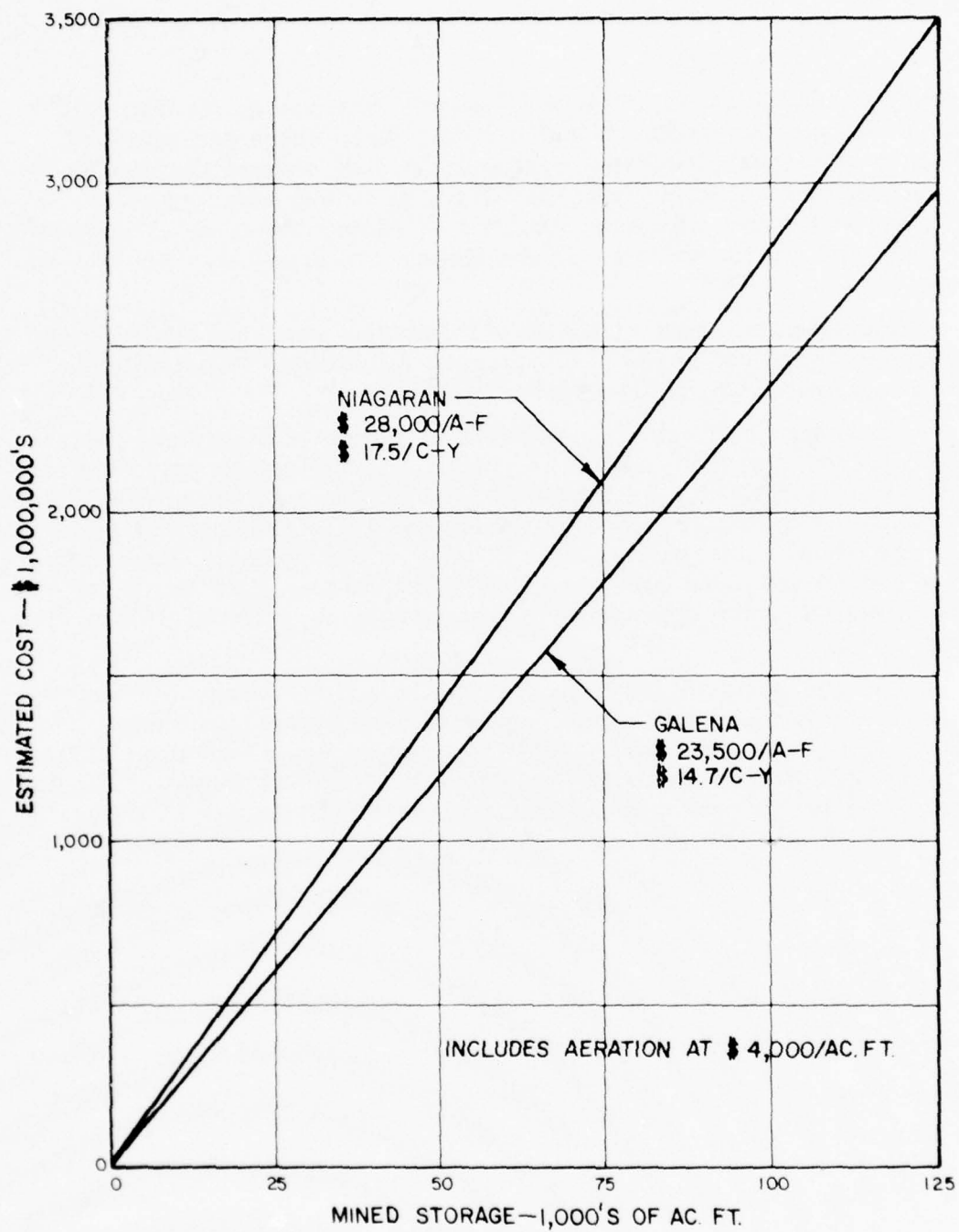
Figure B-VI-D-1
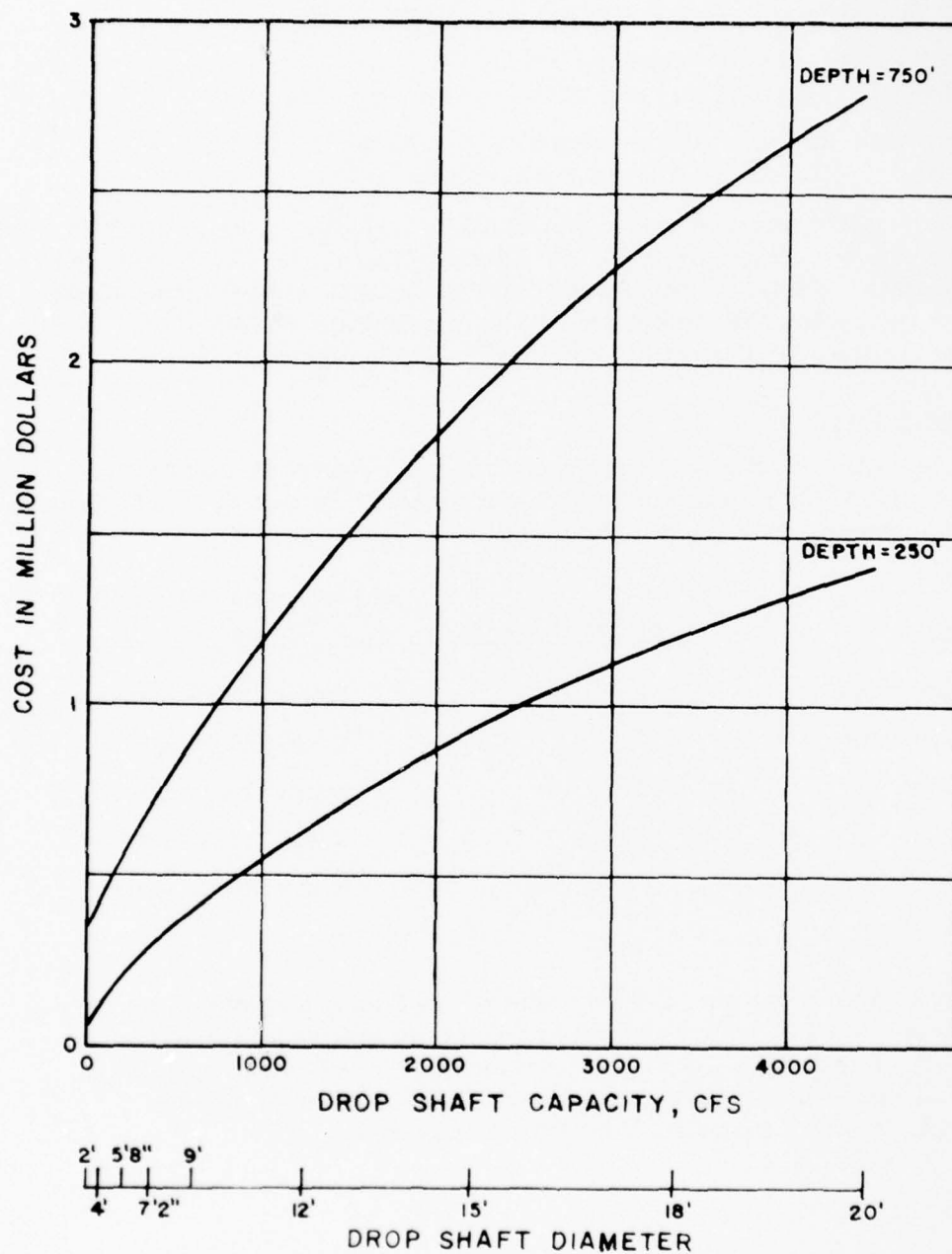
ESTIMATED COST—MINED STORAGE

Figure B-Ⅵ-D-2
COST OF DROP SHAFTS

B-VI-D-5

Operation and Maintenance Costs

Operation and maintenance costs include such items as Labor and Materials required for regular operation and maintenance of shallow pit storage facilities or mined storage facilities.

Labor costs include such personnel as superintendents, operators, clerks, laborers, electricians and other related tradesmen.

Materials include all the necessary implements to operate all equipment and facilities related to both storage units.

Energy costs are estimated at $0.01 per KWH. Labor and material costs are estimated at 0.5% of Capital Costs plus contingencies (@20% of Capital Cost). Data Annex BA-V-B to this Appendix B shows comparative bases for the selection of the percentage values to be used in the computations of Operation and Maintenance costs.

Replacement Costs

Replacement costs are applicable to such items as aeration, aquifer protection, underdrainage, and grit removal facilities. The following replacement schedule is pertinent:

Component	% of Total Cost	Replacement Required (in years)
Land and Structure	60	None in 50 years
Land and Structure	20	Every 25 years
Mechanical	10	Every 10 years
Other	5	Every 10 years
Other	5	None in 50 years
	100%	

The schedule indicates that 20% of capital cost and contingencies of the pumping station will need replacement at the end of the 25th year of operation. Mechanical components such as pumps, valves, etc., plus other parts will be replaced at scheduled 10-year intervals or four times in the life of the project.

RURAL STORMWATER

General

The following discussion presents the basis of cost for all components of the rural stormwater management systems. Due to the nature of the subwatershed drainage basins described in the basis of design, no single, modular unit which would be universally applicable to the entire rural area is available. For this reason, the basis of cost cannot be applied on a modular basis. However, unit cost figures are available for system components, and can be universally applied to all subwatersheds. Unit cost determinations are, therefore, applied to each of 22 subwatersheds within three major representative watersheds. This allows a total watershed cost to be developed. In general, the only variable unit cost between watersheds is the price of the purchased land, which varies between counties. With this in mind, it is possible to determine a county-by-county, per acre cost for the stormwater management of the rural area by normalizing each total watershed cost by the number of acres in the watershed. This per-acre cost then serves as a multiplier for total system cost when applied on a county-by-county basis.

Capital Costs

Grassed waterways. Cost for the grassed waterway system is on a unit cost basis per linear foot of installed grassed waterway. The cost of installation includes earthwork forming, seedbed preparation, fertilizer, planting, plastic tile drains, and control structures. It is assumed that grassed waterways remain in private ownership. Earthwork costs are calculated at $0.50 per foot. Seedbed preparation, fertilizer and seeding costs are estimated at $40 per acre and converted to a linear foot basis, assuming an average drainage way width of 40 feet. This resulting cost is approximately $0.04 per foot. Tile drain installation costs are estimated at $1.15 per foot. Structure costs are based on a per-unit cost, which is then placed on a per-foot basis. This is accomplished by observing the average slope in representative subwatershed areas and determining the needed spacing between structures to maintain permissible velocities. The spacing is then divided into the structure cost to obtain the per foot cost. This resulting cost is $.31 per foot. Total unit cost on a per-foot basis is, therefore, $2.00.

Retention basin costs. Retention basin capital costs include allowances for land costs, and retention basin costs. Retention basin costs include cost for land clearing and site preparation, excavation and embankment construction, and spillway and control structure costs. Land unit costs vary from county to county, and ranged from a high of $6,300 per acre in Cook County to a low of $2,900 in Porter County. Retention basin unit costs are $375 per acre of subwatershed.

Pumping station to irrigation facility and force main. Pumping station costs are determined by reference to Figure B-VI-C-1, Pumping Station Construction Costs. Pumping station constructions costs include such items as site preparation, pumping station structure, pumping equipment, controls, piping within structure, screening facilities, electrical, heating, and ventilating equipment, and other auxiliary equipment. Force main costs are based on a lineal foot cost. Estimated average construction costs for force mains are shown in Table B-VI-E-1.

Irrigation system. Basis of cost for the irrigation system includes actual equipment costs, necessary earthwork to facilitate equipment installation and certain costs associated with land leasing for the spray irrigation areas. Land for spray irrigation is not purchased. Rig installation costs are estimated at $21 per installed foot of center-pivot lateral. An initial payment is included as an inconvenience payment to the owner of the land under irrigation. This figure is five percent of the per-acre land cost.

Drainage costs. Plastic drainage pipe costs are estimated to be $1.15 per installed foot of pipe. Gravity drain main collectors are costed at the cost per foot of installed pipe. Figure B-VI-D-3 presents installed costs of gravity drainage mains. The drainage and water supply well is a reversible pump facility. Unit costs for the system are $204,000, which includes 1300 feet of cased pipe and the reversible pumping unit.

Power station and transmission lines. A transformer station for a 2,000-acre watershed was found to cost $2.00 per acre, or approximately $26.00 per installed KVA. The cost of buried cable transmission lines from transformer to outer pivot is $3.00 per lineal foot, installed.

Operation and Maintenance Costs

Operation and maintenance costs include labor and materials required for the regulation operation and maintenance of all rural management system components, such as upkeep of grassed waterways, pump stations, and irrigation equipment. Labor costs include three
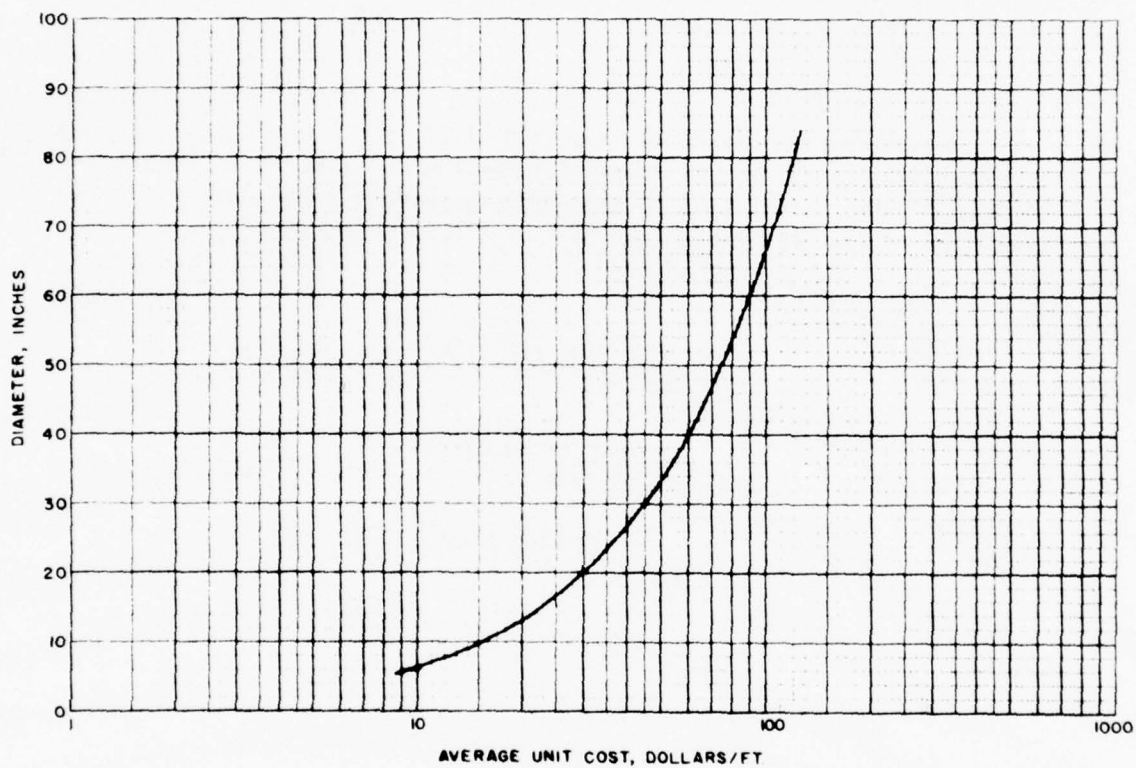
Figure B-VI-D-3

GRAVITY MAIN CONSTRUCTION
COSTS, $/FT.

B-VI-D-9

levels of personnel; semi-skilled, skilled and supervisory. Labor costs used are $10,000, $15,000 and $20,000 per year respectively for those personnel listed above. Materials include all necessary implements for normal operation of the system. Energy costs are estimated at $.01 per KWH. The annual inconvenience factor for land-owners where spray irrigation equipment is used is based on 5 percent of initial land value costs on a per-acre basis.

Replacement Costs

Replacement costs are estimated for the pumping station and center-pivot spray irrigation machines only. Each is estimated to have a 25-year life, and is therefore replaced once during the 50-year life of the system.

VI. COMPONENT BASIS OF COST

E. CONVEYANCE SYSTEMS

CAPITAL COSTS

Capital costs of conveyance systems consist of the cost of (a) force mains, (b) tunnels, and (c) pumping stations.

Force Mains

Capital costs of force mains include such items as air and vacuum relief installations, blow-off installations, and pavement restoration. The costs are expressed in dollars per foot for different diameters of force main. Tabulation of costs of pipes from 10" to 84" in diameter is enclosed. See Table B-VI-D-1 for details.

Tunnels

Tunnels are assumed to be unlined, mole-excavated structures and their costs are based on experience gained by the City of Chicago over the past few years. Three such tunnels have recently been constructed. These are:

1. Lawrence Avenue Tunnel, 5.5 miles long, upper diameter 12 feet, lower diameter 17 feet, cost about $10 million, including allowance for concrete lining.

2. Crawford Avenue Tunnel, 3.5 miles long, diameter about 16 feet, cost about $7.5 million, including concrete lining.

3. Forty-seventh Street Tunnel, 3.5 miles long, diameter about 16 feet, cost about $7.5 million, including concrete lining.

All three of these tunnels were let by competitive bidding with construction to include concrete lining. After experience with the "mole" construction it was decided to eliminate the concrete lining as the bored hole proved to be smooth and strong. Both hydraulic capacity and storage capacity would have been reduced by the concrete lining. Infiltration into the tunnel was found to be controllable through grouting at selected locations, and roof spalling was found to be almost non-existent.

Table B-VI-E-1

ESTIMATED AVERAGE CONSTRUCTION COSTS
FOR FORCE MAINS

Diameter Inches	Average Unit Cost $/ft.
10	13
12	15
14	17
16	20
18	22
20	25
24	30
30	38
36	45
42	53
48	60
54	68
60	73
66	80
72	86
84	99

Present costs for unlined mole-bored tunnels range from $200 per foot for a 10-foot diameter, and $300 per foot for a 16-foot diameter up to $1,000 per foot for a 35-foot diameter. These figures correspond to $1.50/cu. ft. for a 16-foot diameter and $1.00/cu. ft. for a 35-foot diameter. The cost curve for an unlined mole tunnel is enclosed as Figure B-VI-E-1.

Tunnel drop shaft costs are estimated from the cost curve presented in Appendix B, Section VI-D, Figure B-VI-D-2.

Pumping Stations

Pumping station construction costs include such items as site preparation, the pumping station structure, pumping equipment, controls, piping within the structure limits, screening facilities, electrical, heating and ventilating, and other auxiliary equipment. A construction cost curve was prepared for estimating costs of pumping stations. The curve contemplates heads of 100-feet or less. If higher heads are encountered a multiplier is used. This cost curve is shown as Figure B-VI-C-1 in Appendix B, Section VI-C.

Operation and Maintenance Costs

Operation and maintenance costs include labor and material required for regular operation and maintenance of pressure conveyance lines, tunnels, and pumping stations. Cost of power required to run the pumping station is also included. Labor costs include salaries for superintendents, operators, clerks, laborers, electricians and other tradesmen. Materials include all the necessary implements for normal operation of the system. Energy costs are estimated at $.01 per KWH. Labor and material costs are estimated at 0.5% of capital costs plus contingencies (at 20% of capital cost). Data Annex B, Section V-B, shows a comparative basis for the selection of the percentage value to be used in the computations of Operation and Maintenance costs.

Replacement Costs

Replacement costs are only applicable to pumping stations. Tunnels and pressure lines are estimated to last the life of the project and have no replacement factors. The following replacement schedule is pertinent:
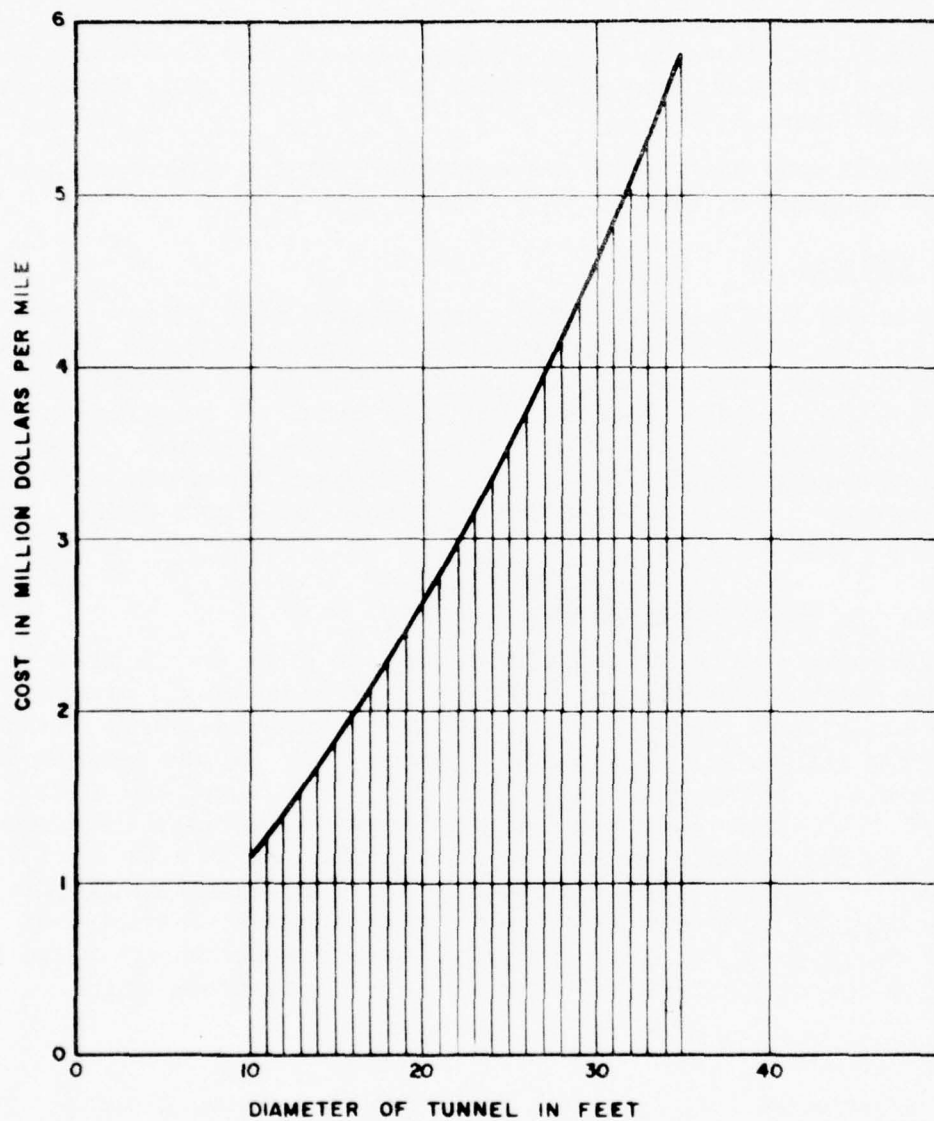
Figure B-VI-E-1

COST CURVE FOR UNLINED
MOLE TUNNEL

Component	% of Total Cost	Replacement Required in Years
Land and Structure	60	None in 50 years
	20	Every 25 years
Mechanical	10	Every 10 years
Other	5	Every 10 years
	5	None in 50 years
TOTAL	100	

The schedule indicates that 20% of capital cost, plus contingencies at 20%, of capital cost of the pumping station will need replacement at the end of the 25th year of operation. Mechanical components such as pumps, valves, etc., plus other parts, will be replaced at scheduled ten-year intervals or four times in the life of the project.

VI. COMPONENT BASIS OF COST

F. ROCK AND RESIDUAL SOIL MANAGEMENT SYSTEMS

The costs associated with the selected management and transportation methods are estimated from available sources, including reports containing applicable information and interviews with people in the materials and transportation fields. The resulting basis of cost information presented here is not detailed but constitutes a best attempt to assign reasonable costs to an enormous materials-management program.

MOUNTAIN LANDSCAPE

The costs of management for this option are broken into three different categories because of differences in the final use and mode of transportation for materials. The categories are (1) overburden and mined rock from the McCook-Summit storage basin (2) moled rock from the urban area, and (3) overburden, mined rock and moled rock in rural and suburban areas.

Overburden and Mined Rock from the McCook-Summit Storage Basin

The cost for management of this material is based upon crushing the rock, loading the rail cars, assembling unit trains of cars with 100-ton capacities, unloading the cars by rotary dumping, and placing the rock at the landfill site. The breakdown, including capital and interest is as follows:

Operating costs for crushing and loading	$0.14/ton
Operating costs for rail haul (20 miles one-way)	0.44/ton
Operating costs for unloading and placement	0.19/ton
Capital and interest	0.36/ton
TOTAL	$1.13/ton

A cost of $1.13/ton is used to obtain the total cost of managing all of the rock and overburden from the McCook-Summit storage basin.

Moled Rock From The Urban Area

It is assumed that one-third of this rock is to be used in the vicinity of its origin. The cost for loading (crushing is not required for moled rock), transport and placement is estimated to be $0.50/ton by using truck transport of less than 5 miles in city driving. For the remaining two-thirds of the rock, the same $0.50/ton is assumed to transport the rock to a rail loading station. The rail loading, transport, unloading, and placement is then estimated to be the same $1.13/ton as is estimated for rock from the McCook-Summit site. It is assumed that the savings from elimination of crushing will be offset by smaller volumes and longer hauls. Thus, for each ton of moled rock produced in the urban area, an average cost is:

$$1/3 \ (0.50) + 2/3 \ (0.50 + 1.13) = \$1.26/\text{ton}$$

An average cost of $1.26/ton is used to obtain the total cost of managing all of the moled rock from the urban area.

Overburden, Mined Rock And Moled Rock In The Rural and Suburban Areas

All of this material is assumed to be used in landscaping open space. Transport is by truck and the distance varies for different locations. A unit cost of $0.75/ton is assumed to apply to all of the materials. This assumes a haul distance of 10-15 miles for overburden and moled rock and somewhat less for mined rock, where crushing is required.

RECREATIONAL ISLANDS

The costs of management for this option are broken down into the same categories as for the mountain landscape. Cost estimates for each of the categories are obtained here.

Overburden And Mined Rock From The McCook-Summit Storage Basin

The components of the cost for management of these materials include loading the barges, barge transport, and unloading the barges. It is also assumed that the material cannot be merely dumped into the lake, but that a wall of steel sheetpiling is constructed around the perimeter of the island prior to unloading. The cost of this sheetpiling is included with that of the material management.

The costs for barge transport are as follows:

Loading of barge	$0.12/ton
Barge transport	0.50/ton
Disposal	0.15/ton
TOTAL	$0.77/ton

The cost of the perimeter wall is estimated by assuming the islands are one-half long and one-eighth mile wide. Thus, 1-1/2 miles of wall will enclose 40 acres of land. A cost per foot of wall is estimated at $650.

Total cost of wall = 1-1/4 x 5,280 x $650 = $4,290,000
Volume of material = 40 acre x 43,560 ft.^{2} x 40 ft. thickness = 2,580,000 cu. yd.
Approximate weight of material = 5,000,000 tons.

To include an allowance for dewatering and placement of large rocks for wave protection, a cost of $1.00 per ton is used as a site preparation cost. Thus, the total cost of management to construct recreational islands is $1.00 + $0.77 or $1.77/ton.

Moled Rock From The Urban Area

The same assumption is made with this material as was in the mountain landscape option. A cost of $0.50/ton is used for truck transport to a nearby landscape or to a barge loading point. Two-thirds of the rock then is transported to the islands at an additional $1.77/ton. Thus, the total unit cost is

$$1/3 \ (0.50) + 2/3 \ (0.50 + 1.77) = 1.69$$

Overburden, Mixed Rock And Moled Rock In The Rural And Suburban Areas

A cost of $0.75/ton is assumed for managing these materials, as in the mountain landscape option.

MOUNTAIN LANDSCAPE WITH COMMERCIAL USE OF ONE-HALF OF THE ROCK FROM THE MC COOK-SUMMIT SITE

The costs of management for this option are assumed to be identical with those for the mountain landscape option, except for the material that is stockpiled for commercial use.

Rock For Commercial Use

Rock prices at the quarry generally range from $1.00 to $1.25/ton when there is no shortage of supply. The costs incurred in mining rock for the storage basin are the same as those normally incurred in mining, except that an additional cost for transport to the stockpile area and for storage must be added. These costs are assumed to be in the range of $0.50 to $0.75/ton, allowing a recovery of $0.50/ton for the one-half of the rock which is used commercially.

This is only a very crude estimate of the recovery that may be gained from sale of the rock, but the variability of the market and the intricacies of negotiations that would undoubtedly be required for this option preclude a more precise estimate.

VI. COMPONENT BASIS OF COST

G. REUSE SYSTEMS

CAPITAL COSTS

Capital costs of water reuse systems consist of the following components:

1. Pumping stations and pressure lines for potable water systems.

2. Pumping stations and pressure lines for recreation and navigation systems.

3. Two pumping stations for use at O'Brien locks and Chicago Harbor locks to transfer lockage flows from the river system to locks. This is to prevent any diversion of Lake Michigan water to the river system. The lock pumpage unit also includes a bubbler system to further prevent intermixing of lake water and river water. Cost of the two pumping stations for O'Brien Locks and Chicago Harbor locks was obtained by using a Pumping Station cost curve. Cost of the bubbler system includes such items as air compressors, air transfer lines, and air manifold pipe at the bottom of channel. Costs were figured on a lump sum basis of $200,000 per each installation. There are four air bubbler systems, two at each lock.

4. Reuse return flow tunnels and pumping stations to return and distribute flows from the sites in the land treatment alternatives.

Capital Costing Information

Costing of force mains, pumping stations, and tunnels, as well as respective cost data including cost curves and tables, is presented in Appendix B, Section VI-E.

Operation and Maintenance

For a discussion on Operation and Maintenance Costs as well as Replacement Costs refer to Appendix B, Section VI-E.

VI.   COMPONENT BASIS OF COSTS

H.   SYNERGISM SYSTEMS

Some of the facilities of the land treatment system can be used jointly for water resource management and electric power generation.   The cost benefits associated with using storage lagoons for pumped storage and waste heat dissipation facilities are described in the following section.

POWER STATIONS

The storage lagoons required for the land treatment system can be used as cooling ponds for the dissipation of waste heat created by the generation of power at power stations.   This cooling can be accomplished without interrupting the storage function of the lagoons.

The unit cost for the construction of a power station cooling pond has been given as $12.40 per kilowatt of production capacity, including an allowance for contingencies and engineering $\underline{1}/$.   Thus, if the total additional energy requirements of the year 2020, amounting to 55,00 MW for the C-SELM study area, are supplied by power stations at land treatment sites, the total construction cost for cooling ponds that could be saved amounts to 682 million dollars.

The annual operation and maintenance cost for this cooling water application has been estimated at 0.1 percent of the construction cost or $0.7 million per year.   This cost includes the labor and the necessary materials and supplies for the system.   There are no replacement costs involved.

With the integration of the power generating system into the land treatment system, speedier environmental approvals for the power generating facilities can be expected since the objectionable disposal of waste heat into natural waters can be avoided.   Assuming that the power station can be constructed two years ahead of the normal schedule, and that the invested capital cost is $300 million for a 1,000 MW station and that there is a net after-tax incremental return of 3 percent over alternative investment opportunities on this investment, then the value of an earlier production date will be $9 million dollars each year or $18 million dollars for the two years.

The total power cost for the main wastewater lift station, for the aerated lagoon and for the irrigation drainage system in the year 2020 will be $65.6 million, based on a current unit cost of $0.01/KWH. This unit cost for electricity can be expected to decrease if a power station is installed at the land treatment site since no major transmission line would be required. Projecting a 50 percent reduction in power cost, the savings on O & M costs for the land system can then be computed as $32.8 million/year. The cost for distribution and transmission of electrical energy amounts to 50 percent of the total cost of electrical energy. [2]

The above computations are based on the assumption that the land required for a cooling pond not associated with the land treatment storage lagoon can be readily acquired with no undue delays. If the land cannot be acquired as easily as desired, the power company could feel compelled to use cooling towers. The unit cost for the construction of a cooling tower is $15.20 per kilowatt of production capacity including an allowance for contingencies and engineering. If the total additional energy requirements of the year 2020 for the C-SELM study area were supplied by a power station at the land treatment site the total cooling pond construction cost that could be saved is $836 million dollars.

The annual O & M cost for the cooling tower is 1 percent of the construction cost or $8.4 million dollars. This cost includes labor, materials and supplies, and power costs.

The total saving which might result by integrating the power station into the proposed land treatment system are summarized and presented in Table B-VI-H-1.

PUMPED STORAGE SYSTEM

The dead storage space of the storage lagoons in the land treatment system of the C-SELM study can be used as the upper reservoir of a pumped storage power generating system.

The unit cost of a storage lagoon of the size required for this application can be estimated at $700 per MG stored. Thus, if all the wastewater from the C-SELM study area were to be treated in a land treatment system, and the full 10,000 MW of probable peaking capacity could be installed and 35,000 MG of the dead storage could used as an upper reservoir, the associated cost benefit from using this portion of the dead storage in the storage lagoon for the upper reservoir of a pumped storage system would be 24.3 million dollars. The annual operation and maintenance cost is estimated to be 0.1% of the construction cost or $0.2 million per year.

Table B-VI-H-1

SYNERGISTIC COST BENEFITS

Item	One Time Cost (million \$)	O & M (million \$/Yr.)
Cooling System	682[a]	0.7[a]
	836[b]	8.4[b]
Upper Reservoir	24.3	0.2
Earlier Production	990	-
Power Cost	-	32.8
TOTAL	1,696.3[a]	40.7[a]
	1,850.3[b]	48.4[b]

[a]The cost of using a storage lagoon as the cooling system.

[b]The cost of using a cooling tower as the cooling system.

BIBLIOGRAPHY B-VI-H-A

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2.   Golze, A. R., "Impact of Urban Planning on Electric Utilities", Professional Engineer, March, 1973.

VI. COMPONENT BASIS OF COST

I. NON-STRUCTURAL SYSTEMS

The nature of non-structural considerations deals with practices and programs where costs are peripheral to the principal thrust of the study. Cost analysis and evaluation typically receives less emphasis in these investigations.

As discussed in Appendix B, Section IV-I, costs incurred in non-structural pursuits may be considered additive to any of the alternative approaches selected in the wastewater management program.

TECHNICAL APPENDIX B

IMPACTS OF MANAGEMENT SYSTEMS

VII. IMPACTS OF MANAGEMENT SYSTEMS

A. INTRODUCTION

The purpose of this section is to present a varied number of impacts resulting from the construction and operation of AWT management systems. These impacts include projected chemical and energy demands for the wastewater treatment facilities, the land acquisition and people displacement requirements for the construction of the various components of the management systems, and the labor requirements for the operation and maintenance of these management systems.

Projected impacts on the air resource from the operation of wastewater treatment facilities are also presented, together with an analysis of impacts on the weather and the rural groundwater resulting from the operation of the land treatment system.

The performance of the wastewater management systems is examined under varied operating conditions to indicate the reliability of such systems. Also the components of the management systems are viewed as to their adaptability for other uses if new technological advances replace their need as a treatment unit. Finally, the management systems are examined concerning the disruption of existing land uses necessitated by the construction of such systems.

The impacts, together with the cost and system performance evaluations, are presented and utilized in Appendix D for a comparison of alternative wastewater management systems which are designed to meet the ultimate water quality goals of this study.

VII. IMPACTS OF MANAGEMENT SYSTEMS

B. IMPACTS ON RESOURCES

CHEMICAL AND ENERGY REQUIREMENTS

For the achievement of existing or NDCP effluent standards, treatment facilities consume extensive amounts of chemicals and energy. Presented in Table B-VII-B-1 are the chemical and primary energy requirements of conventional and AWT facilities for treating one MG of influent wastewater flow. In this table, the requirements associated with the disposal of sludge resulting from the treatment of one MG of wastewater are also included. These unit resource consumption figures are obtained from the detailed cost analyses for the regional treatment and sludge management systems presented in Appendix B, Sections VI-A and VI-C respectively. The figures presented in this table take into account resources consumed in treating recycled flows. The chemical and energy requirements associated with the conveyance, stormwater management and reuse systems are not presented in this table since they are common to the various AWT treatment systems. The chemical and fuel impacts of these systems are insignificant compared to the regional treatment system while their electrical requirement is some 25% of that of the regional treatment systems.

Primary energy requirements refer to the actual electrical and fuel demands of treatment facility and sludge management. Fuel demands are presented in terms of natural gas equivalents since natural gas is the most acceptable fuel from an air pollution point of view. Although alternative fuels exist, they either require extensive pollution control facilities or their manufacture is in the research and developments stage. The secondary energy requirements are presented in Table B-VII-B-2. These energy needs refer to the electrical and natural gas equivalent requirements for the manufacture of the various chemicals presented in Table B-VII-B-1. A natural gas equivalent credit is given to the conventional, advanced biological and land treatment systems as shown in Table B-VII-B-2. This credit is given since the agricultural use of nitrogen fertilizer, which requires the consumption of a natural gas-equivalent fuel, is relieved to the extent of the nitrogen applied by the sludge utilization and wastewater irrigation programs. A com-

Table B-VII-B-1

RESOURCE CONSUMPTION OF REGIONAL TREATMENT AND SLUDGE MANAGEMENT SYSTEMS

TREATMENT SYSTEMS

Resources	Conventional	Advanced Biological	Physical-Chemical	Land Treatment
Chemicals				
Activated Carbon (#/MG)	–	34	68	–
Aluminum Sulfate (Liquid) (#/MG)	–	117	117	–
Chlorine (#/MG)	33	33	33	33
Clinoptilolite (#/MG)	–	–	160	–
Lime (#/MG)	–	1,284	1,764	–
Methanol (#/MG)	–	334	–	–
Polymer (#/MG)	–	1	1	–
Sodium Chloride (#/MG)	–	–	630	–
Primary Energy Requirements				
Electrical 1,000 BTU/MG[a]	3,100	10,400	9,000	22,400[b]
Fuel 1,000 BTU/MG[c]	100	28,100	52,800	100

[a] Electricity @3414 BTU/KWH. Includes sludge management electrical requirements based on agricultural utilization of sludge.

[b] Includes electrical requirements of lifting the reclaimed water from reuse tunnels to C-SELM surface streams.

[c] Fuel is taken as a natural gas-equivalent with a fuel value @1000 BTU/cubic foot. Includes sludge management fuel requirements based on agricultural utilization of sludge.

Table B-VII-B-2

SECONDARY ENERGY IMPACTS OF REGIONAL TREATMENT AND SLUDGE MANAGEMENT SYSTEMS

TREATMENT SYSTEMS
SECONDARY ENERGY REQUIREMENTS (1,000 BTU/MG)^a

CHEMICAL	Conventional	Advanced Biological	Physical-Chemical	Land Treatment
Chlorine (electrical)	180	180	180	180
Methanol (natural gas)	-	4,510	-	-
Lime (natural gas)	-	-	3,750	-
Nitrogen Fertilizer (natural gas)	(1,690)^b	2,730)^b (1,690)^b	-	(9,590)^c
Total Secondary Electrical Requirements	180	180	180	180
Total Secondary Natural Gas- Equivalent Requirements	(1,690)	5,550	3,750	(9,590)
Crop drying (gas)	--	--	--	1,000

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^{a}Electricity @3,414 BTU/KWH. Natural Gas-equivalent fuel value @1,000 BTU/cubic foot.

^{b}This energy saving is based on sludge application of 515 pounds available nitrogen/acre/year and a natural gas-equivalent requirement of 140×10^6 BTU/ton of nitrogen fertilizer.[1/] This energy requirement refers to the manufacture of nitrogen fertilizer. The total energy requirement, including raw resource recovery, transportation etc., may be twice the amount shown in the table.

^{c}Based on sludge application of 515 pounds available nitrogen/acre/year and a wastewater application of 500 pounds available nitrogen/acre/year and a natural gas-equivalent requirement of 140×10^6 BTU/ton of nitrogen fertilizer.

parison of these tables indicates that the secondary natural gas requirement for the advanced biological system is some 20 percent of the primary requirement. For the physical-chemical system this secondary natural gas impact is equal to approximately 7 percent of the primary requirement.

For the conventional and advanced biological treatment systems, some 3.7 million BTU/MG of methane gas is produced and consumed by heated anaerobic digesters. It is further estimated that 4.4 million BTU/MG of excess methane gas is produced in the same digestion facilities. No credit is taken for this excess gas production in Table B-VII-B-1 since a high degree of reliability is difficult to maintain due to process susceptibility to biological upsets.

An energy credit is not given to the physical-chemical sludge management system even though energy requirements for the manufacture of lime are relived. This credit is not given since all types of sludge possess inherent capabilities to control soil pH. In the physical-chemical sludge, the lime content controls the soil pH. In the other types of sludge, sufficient buffer capacity is provided through the organic nature of the sludge to control soil pH for agricultural utilization.

The chemical and energy demands of the treatment facilities impact mainly on the availibility of the chemical and energy resources. Of prime concern at the present time is the limited supply of natural gas and electrical generating facilities. Natural gas is utilized as a fuel source for the AWT regeneration processes and the ammonia stripping process. Alternative fuel sources, such as coal or fuel oil would have impacts on the air resource due to additional sulfur dioxide emissions. Natural gas is also utilized in the manufacture of methanol. If this impact on the natural gas supply is significant, then an alternative more costly carbon source would have to be substituted for the advanced biological denitrification process. Presently in the research and development stage is the synthetic manufacture of natural gas through a coal gasification process. The development and implementation of this process may alleviate the impact of the treatment facilities natural gas demands on available natural gas supplies.

The impact of the electrical requirements of the treatment facilities is related to impending decisions on the implementation of nuclear generating facilities. If it is found that the safety provisions and environmental impacts are such that the programmed facilities may be placed
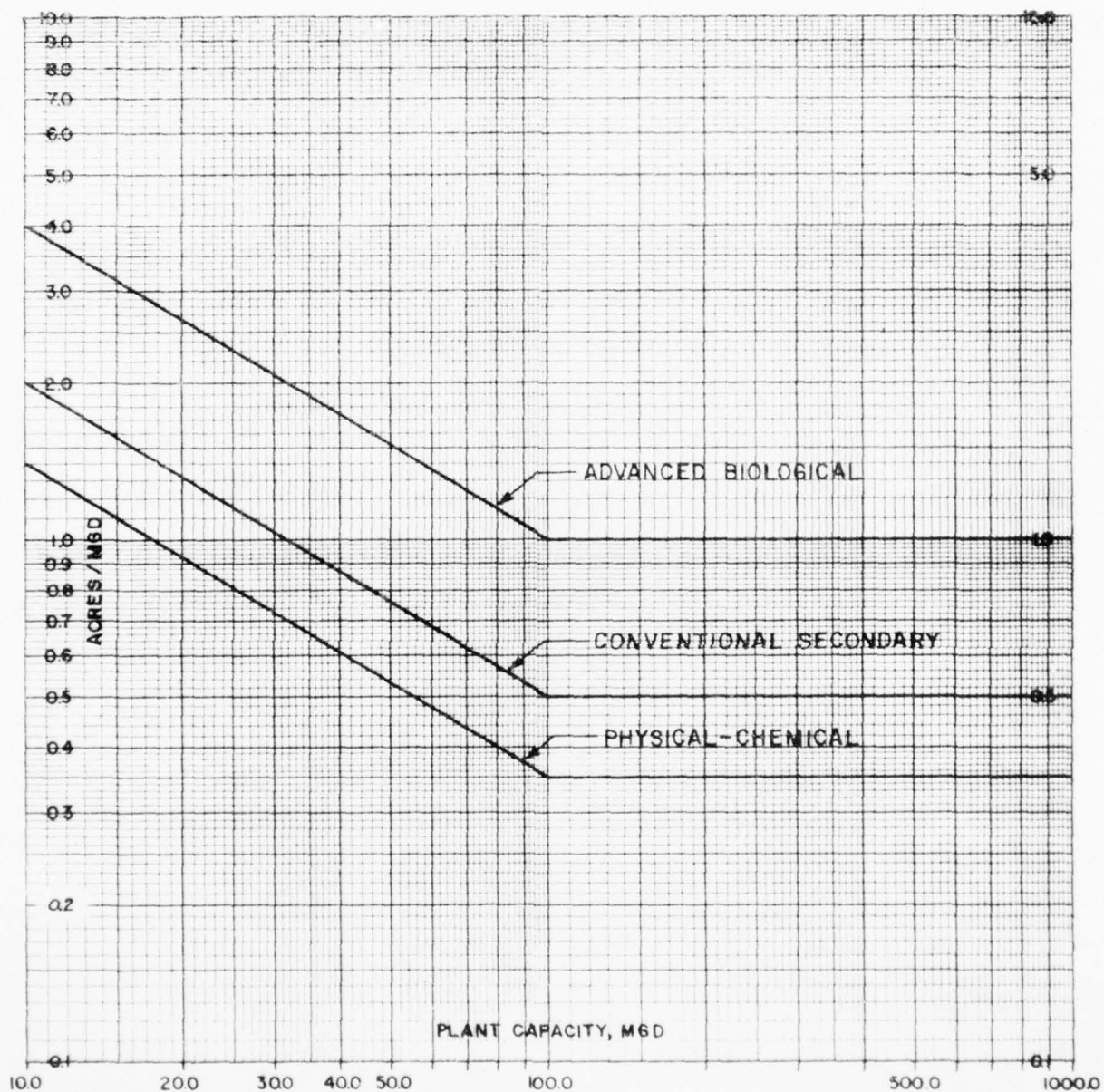
Figure B-VII-B-1

LAND REQUIREMENTS FOR TREATMENT FACILITIES

in operation, then the electrical requirements of the treatment facilities is much less significant than if the programmed nuclear stations are prohibited from coming on-line. As noted in Appendix B, Section IV-H, the land treatment system has the adaptability to incorporate power generating facilities, thus further decreasing the electrical impact of the land treatment system.

LAND DISPLACEMENTS

Introduction

There are a number of factors which are examined in the assessment of land displacement impacts. The particular utilization of the land establishes whether it is leased or purchased. Purchasing of land may have impacts of new land use, people displacements and removal of land from the tax base. Leasing of lands may have impacts on restricting future land use considerations.

The type of land displacement is an important parameter in assessing system impacts. For example, relatively small urban or suburban land displacements may affect the same number of people as that of large rural displacements, due to the varying densities within these land use types.

Treatment Facilities

Treatment plant system. Land requirements for this mode of treatment are based upon actual requirements of existing facilities together with the proposed treatment plant system layouts as presented in Appendix B, Section IV-A. 2/ A curve representing these two sources is presented in Figure B-VII-B-1. This curve yields a land requirement in acres per MGD, for different size plants.

As presented in Figure B-VII-B-1, the physical-chemical treatment system requires the least amount of land. This is due to the fact that the detention times of these treatment components are relatively small and also because a number of the unit processes are amenable to vertically oriented equipment which provide sufficient contact volume while occupying a relatively small area. For the conventional and advanced biological treatment systems, the land requirements increase due to increased detention times of the biological treatment units together with overflow rates necessitating large clarifier areas, and the need for storing on-site sludge volumes during the winter months.

Land treatment system. Land requirements for this mode are based on the modular land treatment design as determined in Appendix B, Section IV-A. The land requirements for the 265-MGD module are 5,600 acres of lagoon land and 26,500 acres of irrigated land. Assuming a 40 percent irrigation land utilization factor, the total land requirement for the land treatment module is some 72,000 acres or an equivalent 270 acres per MGD of influent wastewater flow. Land for storage lagoons and treatment cells, within each modular unit, is purchased in fee simple. Contractual arrangements will be made with local farmers for the utilization of their lands for irrigation purposes. This policy affords minimum disruption to surrounding communities, yet obtains all the land which is essential to the operation of the system. *Associated costs, on a per acre basis, include such items as land* value, dwelling unit value, average farm size, average household size, residential and non-residential density units. The costs were computed for several counties in the C-SELM area. The values obtained are presented in Data Annex B, Section VII-A.

Sludge Management

Agricultural utilization. The land requirement for sludge application is obtained by dividing the total dry tons of disposable sludge by the allowable application rate and adding a 10 percent allowance for buildings, roads, and other access requirements. These application rates and sludge yields are presented in detail in Appendix B, Section IV-C. Utilizing this data, the conventional, advanced biological and land treatment systems require that some 23 acres of sludge disposal land/MGD of treatment plant capacity to be contracted for to accomplish an agricultural utilization of this sludge. For an agricultural utilization of the physical-chemical sludge, approximately 260 acres of land/MGD are required. As in the land treatment and advanced biological treat-

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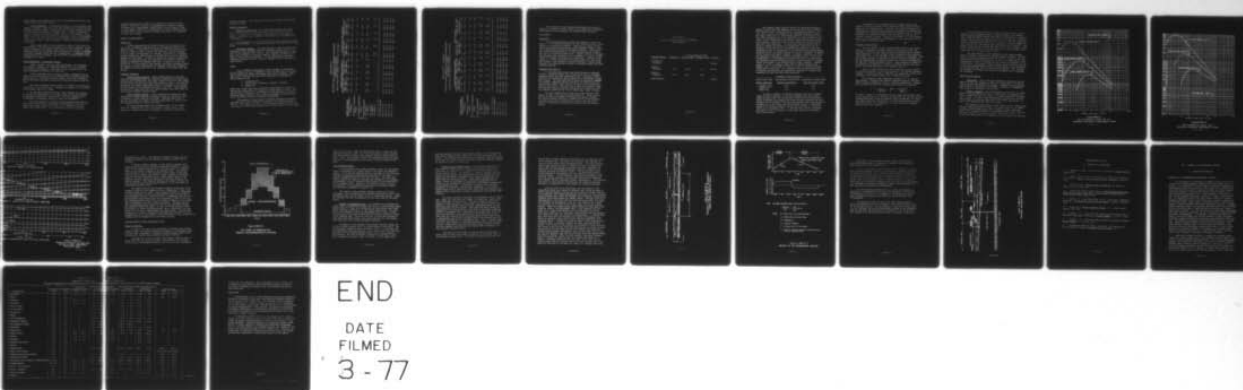FILMED

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ment systems, this sludge land would not be bought, but would be contracted for from the local farmers.

Land reclamation. A methodology similar to the agricultural utilization of sludge is employed in the land reclamation of sludge, but with different application rates. Application rates for each treatment system are given again in Appendix B, Section IV-C. For the conventional, advanced biological and land treatment systems, approximately 3 acres/ MGD are utilized each year since the application is on a short duration basis. Thus over the 50-year life of the system, the land reclamation requirement for these sludge management systems is approximately 150 acres/MGD.

Actual requirements of land reclamation depend a great deal on the character of the land being enriched. It is also an objective of this study to maximize the number of acres which can be reclaimed. More sludge per acre may be required than assumed in this study on the basis of nitrogen loadings during the year of application. If additional applications prove to be necessary, then fewer than 150 acres/MGD can be reclaimed in 50 years.

Land Requirements - Stormwater Systems

Urban, suburban, and rural land requirements, for stormwater and collection systems, are obtainable directly from actual storage, and irrigation requirements, as presented in the component basis design of Appendix B, Sections IV-D and IV-E.

For the collection and conveyance systems, easements are secured in the public right-of-way to construct such facilities below the ground surface. This does not really impact on the land requirement but on the system disruption which will be discussed later in this section.

The urban stormwater storage system is designed to utilize existing quarries or pits excavations. Therefore, the impact of this system on the present land use is slight.

The suburban stormwater storage system requires the purchase of land for the construction of the various types of storage facilities as detailed in Appendix B, Section IV-B. This land requirement is 6 acres/MGD of treatment capacity to treat such stormwater.

The rural stormwater management system requires the purchase of land for the construction of stormwater runoff detention reservoirs together with nearby, contracted-for lands for the irrigation and treatment of the stored runoff. Of the total rural area to be managed,

approximately 5 percent of the land is utilized as storage facilities while an additional 12 percent is programmed for irrigation purposes. Dividing by the corresponding average annual flow so treated, some 15 acres/MGD of land are to be purchased for storage reservoirs while 34 acres/MGD of land to provide irrigation sites are to be contracted for with local farmers.

PEOPLE DISPLACEMENTS

Introduction

People displacements are directly related to the particular function of the management component and to the population density of a particular area. If lands are to be purchased, then the land use character is changed in such a manner that relocation of people residing in these lands is necessitated. For example, wastewater or stormwater storage facilities, by nature, require the displacement of people presently residing on these lands. In order to minimize the number of people displaced, the purchased lands are programmed to be those in which the population densities are presently very low. If lands are to be contracted for, as is the case for the irrigation and sludge utilization areas, the impact of people displacements is minimized since these facilities are designed and laid-out so as to be consistent with present land use patterns.

Treatment Facilities

Treatment plant systems. People displacements for the areas allocated to treatment plants are obtained by multiplying the gross township population densities for each plant location by the new area needed for that plant over and above that which presently exists. As mentioned previously, the people displacement impact is a function of the amount of land needed and the population density of that land. Thus relatively small tracts of urban land may have greater impacts than large tracts of less densely populated suburban lands.

Land treatment systems. People displacements within the land purchased for storage lagoons and treatment cells are obtained by similar methods to those previously described. Although large tracts of land are purchased,,the impact of people displacement is offset by the low population density of these rural lands.

No people displacements are anticipated within the land treatment site irrigation areas due to the modular system design, which takes into consideration present distribution of population centers and

farmland dwellings. Thus irrigation systems are located to avoid such population centers.

Sludge Management

Agricultural utilization. No people displacements are anticipated within the areas of agricultural utilization of sludge since the system is designed to be consistent with present land use patterns. Sludge is applied to lands already in cultivation and consequently free of habitation.

Land reclamation. Reclamation lands are former strip mine areas, which are uninhabited. No people displacements are expected from these lands.

Stormwater systems. The same analysis is used for people displacements for the suburban and rural stormwater storage areas, as is used for the treatment plant system. Total required acreages, by township, are multiplied by the gross population densities to obtain a numercial value of people displaced. For the urban storage and rural stormwater irrigation systems, sites are selected where no people presently reside.

LABOR

Use of different technologies for the treatment of wastewater in the C-SELM area has a significant impact on total labor force which may be required to operate and maintain the various systems. Total labor force is divided into three broad categories of responsibility. These categories are:

      1.   Unskilled labor
      2.   Skilled labor, mechanics, operators, analysts
      3.   Supervisors

With such a labor force there exist great possibilities for on-the-job training and advancement of categories of higher degree of skill. Proper on-the-job training can be beneficial to all employees not only in attaining higher skills but also in the understanding of over-all operation of the entire system of wastewater management.

The measure of labor impact for different treatment technologies for target dates 1990 and 2020 is presented in the following Table B-VII-B-3. The number of persons in each category required per 100 million gallons of wastewater treated per day is the measure of labor impact.

Table B-VII-B-3

PROJECTED LABOR IMPACTS FOR REGIONAL WASTEWATER
MANAGEMENT SYSTEMS

(Values Shown are: Number of Persons per 100 MGD)

System Description	Standard Biological Treatment						Physical-Chemical Treatment					
	Design Year 1990			Design Year 2020			Design Year 1990			Design Year 2020		
	Labor Class			Labor Class			Labor Class			Labor Class		
	1	2	3	1	2	3	1	2	3	1	2	3
Wastewater Treatment Facilities	37.8	47.3	4.8	38.0	47.4	4.9	79.0	140.0	28.0	79.1	140.0	28.0
Wastewater Conveyance System	1.5	4.4	2.2	1.2	3.6	1.8	1.6	4.7	2.3	1.3	3.9	1.9
Stormwater Management System	-	-	-	-	-	-	15.3	31.6	15.7	9.2	19.1	9.6
Sludge Management												
(1) Option 1	6.0	5.3	.5	5.5	4.9	.5	25.8	23.8	2.3	25.2	23.2	2.3
(2) Option 2	6.0	5.3	.5	5.5	4.9	.5	-	-	-	-	-	-
Reuse Systems												
(3) Option 1	-	-	-	-	-	-	1.0	.4	.2	.8	.3	.2
(4) Option 2	-	-	-	-	-	-	1.0	.4	.2	.8	.3	.2
Total Labor Impacts												
With (1) & (3)	45.3	57.0	7.5	44.7	55.9	7.2	122.7	200.5	48.5	115.6	186.5	42.0
With (2) & (3)	45.3	57.0	7.5	44.7	55.9	7.2	96.9	176.7	46.2	90.4	163.3	39.7
With (1) & (4)	45.3	57.0	7.5	44.7	55.9	7.2	122.7	200.5	48.5	115.6	186.5	42.0
With (2) & (4)	45.3	57.0	7.5	44.7	55.9	7.2	96.9	176.7	46.2	90.4	163.3	39.7

B-VII-B-11

Table B-VII-B-3 (Continued)

PROJECTED LABOR IMPACTS FOR REGIONAL WASTEWATER
MANAGEMENT SYSTEMS

(Values Shown are: Number of Persons per 100 MGD)

System Description	Advanced Biological Treatment						Land Treatment					
	Design Year 1990			Design Year 2020			Design Year 1990			Design Year 2020		
	Labor Class			Labor Class			Labor Class			Labor Class		
	1	2	3	1	2	3	1	2	3	1	2	3
Wastewater Treatment Facilities	108.0	160.0	30.0	108.0	160.0	30.0	49.1	27.6	6.2	49.1	27.6	6.1
Wastewater Conveyance System	1.5	4.5	2.3	1.2	3.7	1.9	2.4	7.1	3.6	2.0	5.9	3.0
Stormwater Management System	15.3	31.6	15.8	9.2	19.1	9.6	15.4	31.8	15.9	9.3	19.2	9.7
Sludge Management												
(1) Option 1	4.5	4.1	.4	4.2	3.8	.4	4.1	3.7	.4	4.1	3.6	.4
(2) Option 2	7.5	6.7	.6	6.8	6.1	.6	6.1	5.5	.5	5.7	5.2	.5
Reuse Systems												
(3) Option 1	1.0	.4	.2	.8	.3	.2	6.7	2.7	1.3	5.6	2.2	1.1
(4) Option 2	1.0	.4	.2	.8	.3	.2	6.7	2.7	1.3	5.6	2.2	1.1
Total Labor Impacts												
With (1) & (3)	130.3	200.6	48.7	123.4	186.9	42.1	77.7	72.9	27.4	70.1	58.5	20.3
With (2) & (3)	133.3	203.2	48.9	126.0	189.2	42.3	79.7	74.7	27.5	71.7	60.1	20.4
With (1) & (4)	130.3	200.6	48.7	123.4	186.9	42.1	77.7	72.9	27.4	70.1	58.5	20.3
With (2) & (4)	133.3	203.2	48.9	126.0	189.2	42.3	79.7	74.7	27.5	71.7	60.1	20.4

The construction of the wastewater management system components as presented in Table B-VII-B-3 would also have a significant labor impact on the construction and material-supply industries.

AIR QUALITY

Introduction

All wastewater treatment systems have impacts on the air resource. For conventional biological and land treatment systems, minor impacts on air quality are produced. These are due mainly to localized aerosol affects from biological aerated treatment facilities; these can be effectively controlled by screening procedures. Odor impacts from sludge dewatering facilities or land treatment storage facilities may also occur over short time periods. Lagoon facilities are designed with adequate buffer distances so as to minimize, or "screen", possible adverse air impacts on adjoining rural populations. Presented in this section is a general discussion of air quality impacts for the various AWT systems. Also included are detailed analyses of nitrogen oxide emissions from treatment plant systems, and weather impacts from land treatment facilities.

Treatment Plant Systems

For the advanced biological and physical-chemical treatment systems, significant impacts on air quality occur due to incineration processes, mainly from lime recalcination units. Presented in Table B-VII-B-4 are projected air quality impacts for the wastewater management systems. The particulate data is based on particulate standards for incinerator emissions recently proposed by the Environmental Protection Agency (EPA). These call for 0.03 grain or less particulates/ standard cubic foot.[3/] Both AWT plant systems are designed to meet this proposed particulate standard through the use of presently available pollution control devices such as high-energy venturi scrubbers.

The sulfur dioxide data is based upon actual emissions sampled by the EPA at the Lake Tahoe AWT facility. [4/] These emission levels are below any presently contemplated incinerator standards. The physical-chemical emission is greater than the advanced biological system since only an estimated 10 percent of the raw organic solids are incinerated in the advanced biological lime recalcination process.

Table B-VII-B-4

IMPACTS ON AIR QUALITY BY TREATMENT
PLANT SYSTEMS

Treatment System	Air Pollutant(Pounds/MG)			
	Particulates	Sulfur Dioxide	Nitrogen Oxides	Aerosols
Conventional- Biological	-	-	-	Present
Advanced Biological	4.68	0.058	1.3	Present
Physical- Chemical	6.91	0.86	360	-
Land Treatment	-	-	-	Present

The nitrogen oxide emission data for the advanced biological system is based upon EPA sampling of the Lake Tahoe AWT Facility.[4] This emission level is presently below contemplated standards. However, in as much as the technology for controlling nitrogen oxides is not highly developed, the contemplated standards which reflect existing control technology are not as restrictive as might otherwise be desired. At the other extreme, the nitrogen oxide emissions from the physical-chemical system are some 20-times greater than existing fossil fuel-fired steam generators. The physical-chemical emission data is based on feeding the ammonia air-stripped gas-stream from the ion exchange elution process to the fluidized-bed incinerator of the lime recalcination unit as a component of the fluidizing and oxidizing gases. Assuming 30 mg/l of influent nitrogen in the raw wastewater, 50 percent of this ammonia and organic nitrogen is oxidized to nitrogen gas while the remaining 50 percent is oxidized to nitrogen oxides. This can be accomplished in a fluidized-bed incinerator because of the near reducing conditions that are maintained locally in the bed while the average overall environment of the fluidized-bed is highly oxidative. Approximately 0.17 tons of nitrogen oxides/MG of treated wastewater are emitted. This is equivalent to some ten pounds of nitrogen oxide per million BTU of fuel input. As a reference, the following table refers to existing and proposed new performance standards for nitrogen oxide emissions of fossil fuel-fired steam generators.

Nitrogen Oxide Emissions

Fossil Fuel Fired Steam Generators	Emissions (lb./million BTU) Existing Installations	Emissions (lb./million BTU) New Performance Stds.
– Gaseous Fuel	0.4	0.2
– Liquid Fuel	0.7	0.3
– Solid Fuel	1.4	0.7

In order to obtain a 50 percent reduction in the above nitrogen oxide low-level emissions, it has been estimated that the pollution control facilities will cost an additional 6 percent of the steam generator capital investment and increase the steam generator operating and maintenance costs by 4 percent.[5] Although these limited control technologies are available, it is not evident that the technology exists to significantly decrease the physical-chemical nitrogen oxide emission to acceptable levels and at reasonable costs. A nitrogen oxide analysis concerning the impact on the air resource for large AWT facilities is presented later in this section.

An alternative to incinerating the air-stripped ammonia gas streams would be to simply let the ammonia nitrogen enter the atmosphere from the stripping tower. However, the ammonia nitrogen concentration is in itself an air pollutant and eventually the nitrogen will be washed into surface streams through the hydrologic cycle. Another alternative for controlling nitrogen oxide emissions is the use of a catalytic reduction process. However, this process is in the research stage and cannot presently be programmed for large applications such as the lime recalcination units.

Nitrogen Oxide Analysis

The recalcination of lime sludge, together with any associated organic and nitrogenous material, produces an estimated 0.0067 dry tons/MG of nitrogen oxide (NO_x) for the advanced biological treatment plant technology. For the physical-chemical treatment plant systems, a generation of 1.73 dry tons/MG of NO_x emissions is estimated. This rate of NO_x emission is equivalent to that of 16,000 automobiles for the advanced biological treatment system and 4,200,000 automobiles for the physical-chemical treatment system. These numbers are based on an average car consuming 2.4 gallons of gasoline per day with each gallon of gasoline generating 0.113 pounds of NO_x. [6]

NO_x is a toxic pollutant and the existing air quality standard for continous exposure is less than 0.05 ppm. The following analysis examines a dispersion model to determine the distribution of NO_x concentration on the ground surface from a single point source.

The dispersion model suggested by Gifford [7] [8] is used for this analysis. Assuming that turbulent disperson prevails, the distribution of NO_x concentration follows a Gaussian distribution. Neglecting any buoyancy effect the following basic equation is obtained:

$$C = \frac{Q}{\pi \, \sigma_y \, \sigma_z \, u} \left\{ \exp \; -1/2 \left(\frac{H}{\sigma_z} \right)^2 \right\}$$

in which C equals the concentration of NO_x along the downwind direction at ground level; Q is the NO_x emission rate; σ_y and σ_z are the standard deviations of the distribution of NO_x concentration in lateral and vertical direction, respectively, U is the wind velocity and H is the stack height. The concentration of NO_x can then be computed based on this basic equation.

If the gaseous emissions from the recalcination unit of an advanced biological treatment plant of capacity similar to the present West-Southwest MSDGC facility (860 MGD) are examined, the emission rate of NO_x from the stack will be 0.575 dry tons per day. Under a typical summer condition, with wind velocity of 10 mph, the distribution of NO_x concentration at ground level is examined for stack heights of 100 ft and 250 ft. The results are presented in Figure B-VII-B-2. The results of the analysis indicate that NO_x will not be a problem. Maximum concentration will be 0.03 ppm under the assumed condition provided there are no other sources of NO_x.

The same analysis is also performed for the same treatment plant recalcination unit, but for a physical-chemical facility. The emission rate of NO_x is estimated to be 150 DT/day. Based on the same design condition, with wind velocity of 10 mph, the distribution of the concentration of NO_x is analyzed and the results are presented in Figure B-VII-B-3. The results of this analysis indicate that there will be a wide area approximately 40 miles in length which will be exposed to an unacceptable NO_x level in the downwind direction. The air pollution problem will become more severe for decreased wind or stagnant air conditions.

Land Treatment System

Introduction. Due to the large wastewater application rate designed into the land treatment system (134 inches/year), it is necessary to examine the potential for change in weather trends particularly with respect to precipitation and humidity. Therefore, a weather impact analysis is presented in this section.

Present conditions. The transformation of water from the liquid phase to the gas phase takes place in three forms: evaporation from soils; transpiration from living plants; and direct evaporation from droplets of water. The rate of this transformation depends upon the relative humidity and turbulent diffusivity of the ambient atmosphere, as well as on temperature and mean air velocity.

The incoming solar radiation in the vicinity of the C-SELM study area has sufficient potential energy to evaporate more than 30 inches of water annually based upon a lake evaporation model, while the annual rate of evapotranspiration for agricultural land in the study area has been estimated to be on the order of 20 inches or more. As a conservative upper limit for the purposes of this analysis, the proposed
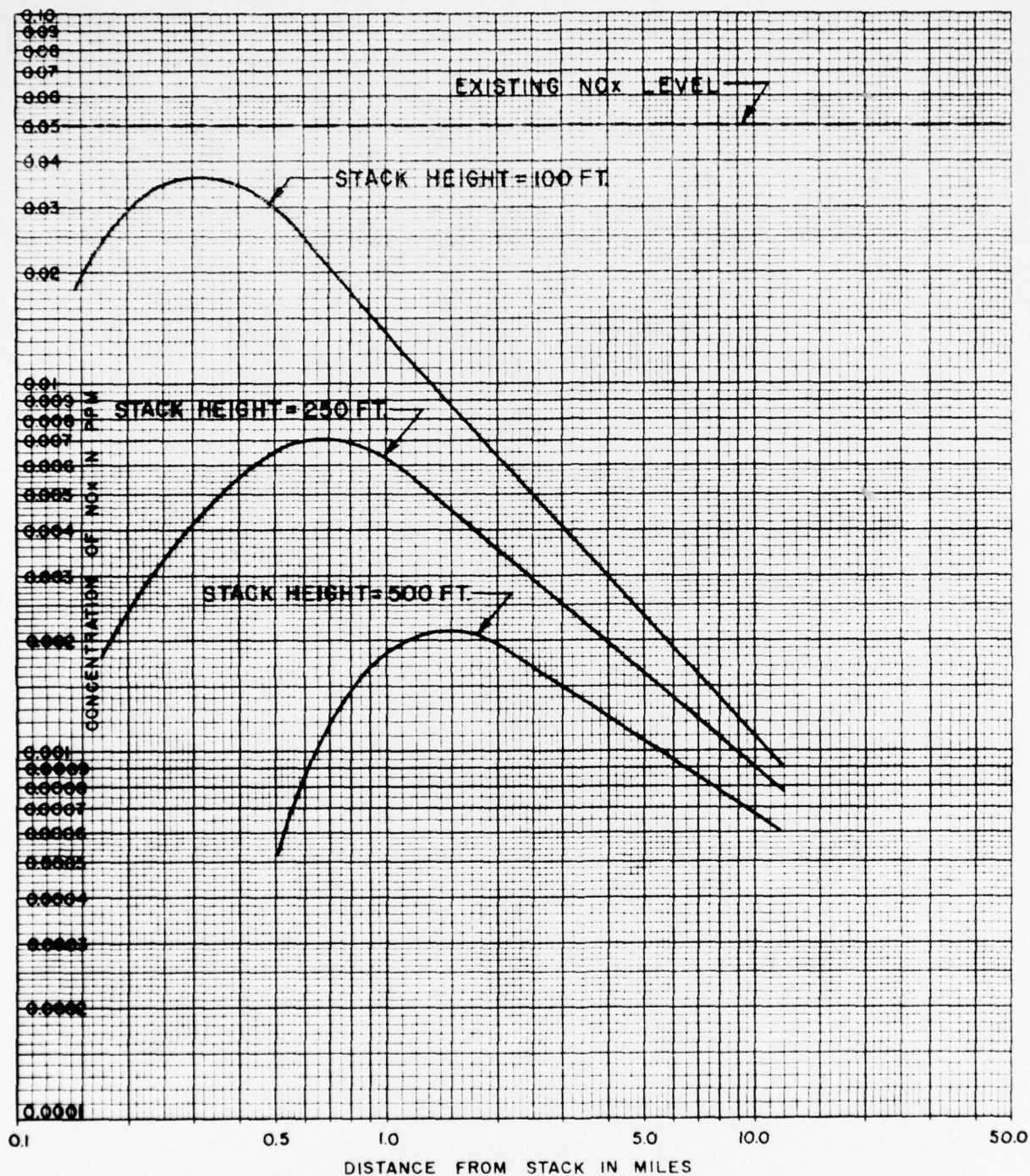
Figure B-VII-B-2

NOx DISPERSION CURVE FOR AN
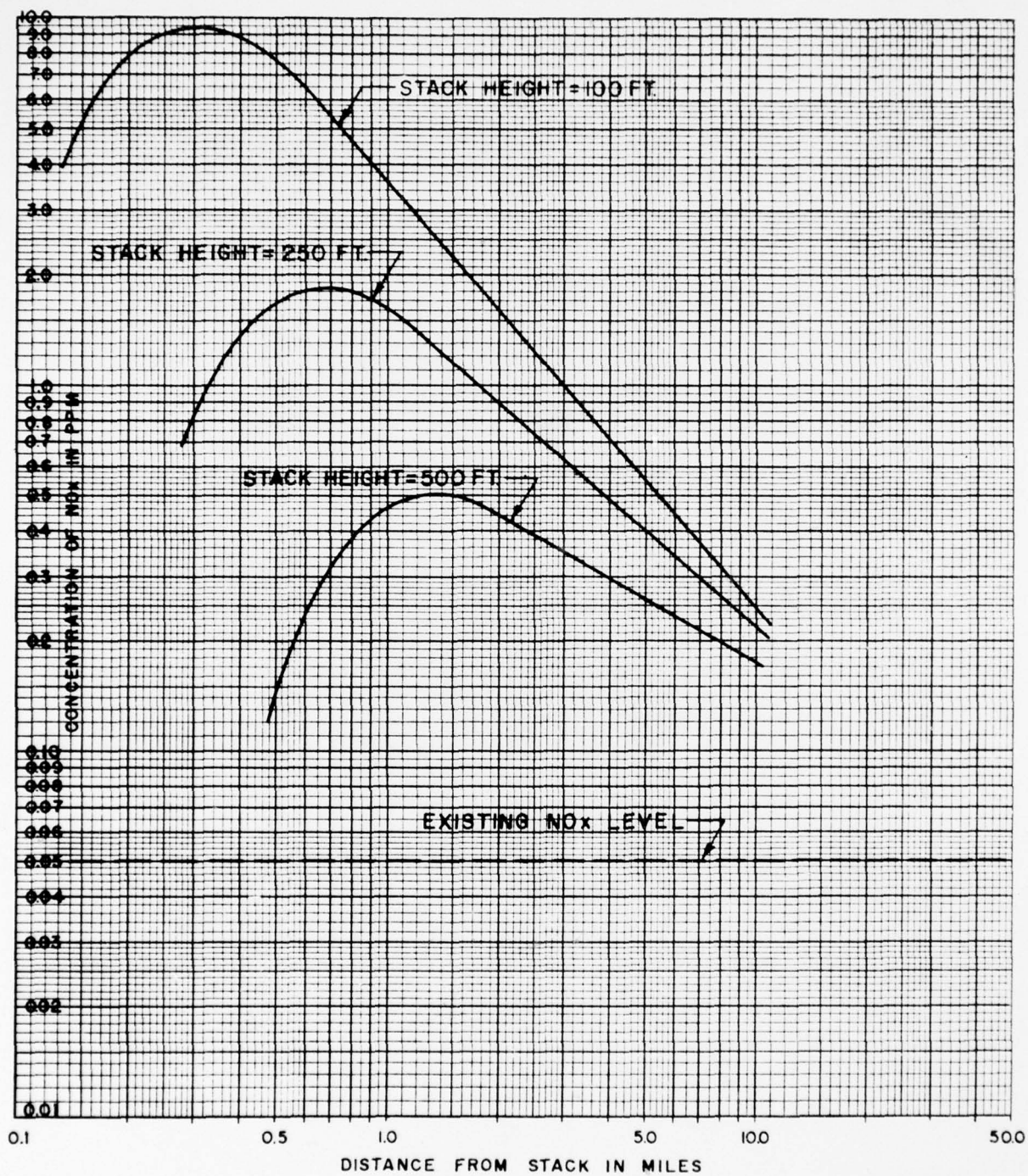ADVANCED BIOLOGICAL TREATMENT PLANT

B-VII-B-18

Figure B-VII-B-3

NOx DISPERSION CURVE FOR A
PHYSICAL–CHEMICAL TREATMENT PLANT

B-VII-B-19

land treatment irrigation system might apply sufficient water to the agricultural lands involved to increase this agriculture rate of evapotranspiration to a level equivalent to that of lake evaporation.
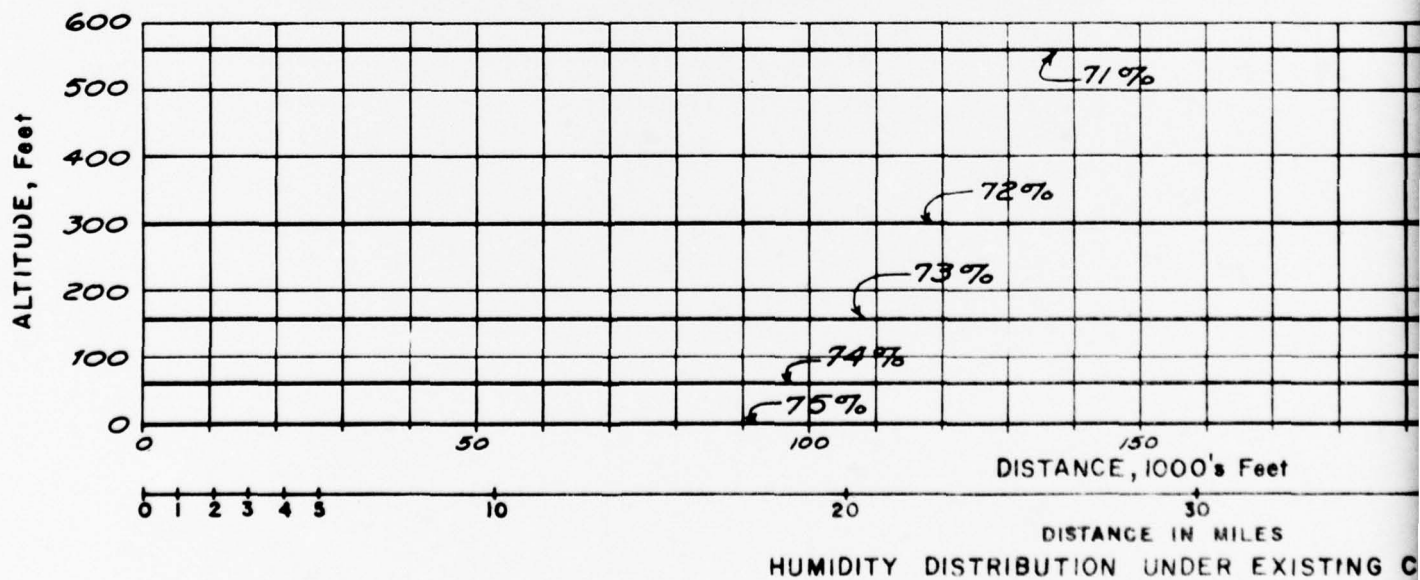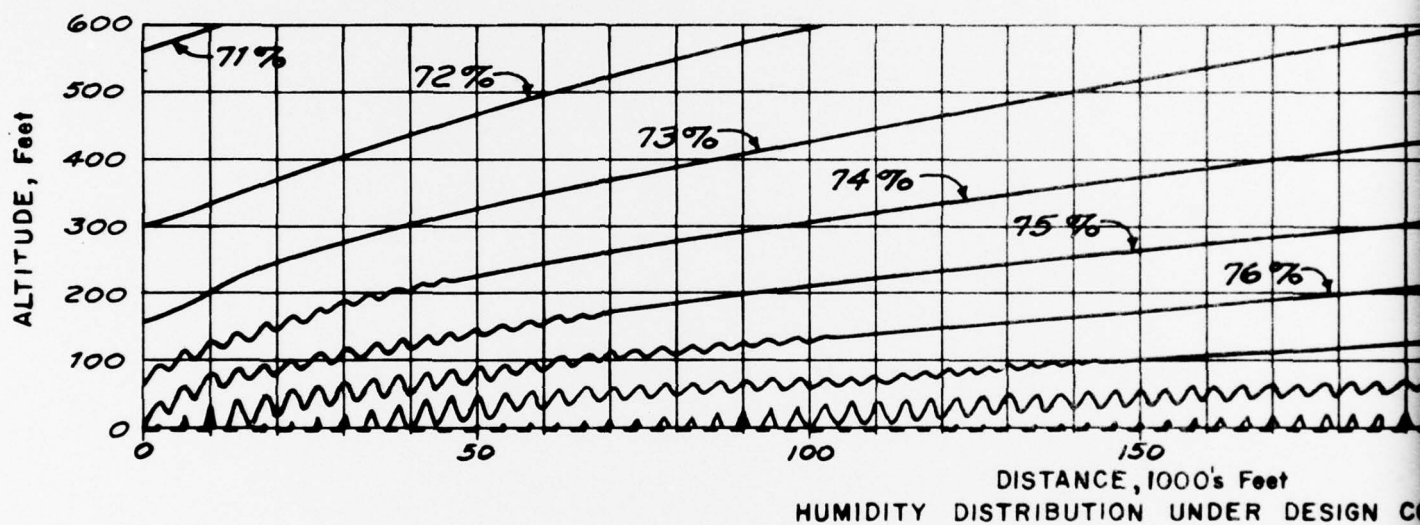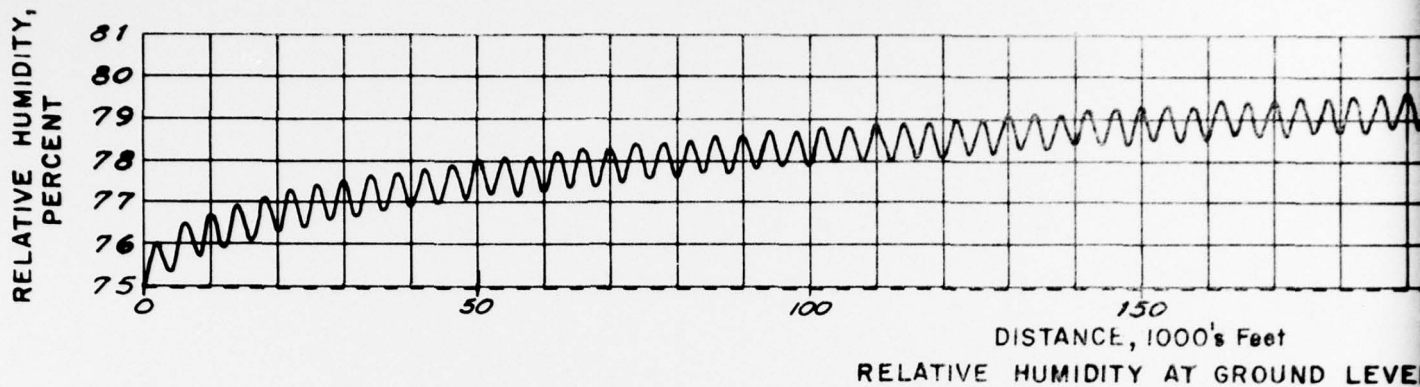
Two critical components of weather which are effected by an increase in the rate of evapotranspiration are humidity and the amount of rainfall. Both of these are considered in this investigation.

Irrigation simulation model. In order to predict the impact of irrigation on the local humidity, a two dimensional, mathematical diffusion model was developed. The following assumptions were made for this analysis:

1.   The irrigation units are 2,000 ft. wide strips of infinite length.

2.   Between two units of irrigation is a 2,000 ft.-wide buffer strip of infinite length.

3.   The gross width of the irrigation area is more than 50 miles.

4.   The rate of evapotranspiration from the irrigation unit is equal to the rate of evaporation expected from a lake surface under the same weather conditions.

5.   The atmospheric temperature change due to the increase in the rate of evapotranspiration is not included for reasons of simplification and also because it is a conservative assumption.

By applying the typical summer conditions for the C-SELM study area of 75°F temperature, 30 mph wind velocity and 75% relative humidity in the upstream region of the irrigation area, the humidity distributions in the vicinity of irrigation area are obtained. The details of the methodology employed in this analysis are attached in Data Annex B, Section VII-B and the results of this analysis are shown in Figure B-VII-B-4.

The analysis indicates that the major impact of the increased rate of evapotranspiration on humidity would occur near ground level and that the maximum increase in humidity is about 5% (from 75% to 80%) when the irrigation area extends in the direction of the prevail-

RELATIVE HUMIDITY AT GROUND LEVE[L]

HUMIDITY DISTRIBUTION UNDER DESIGN C[ONDITIONS]

HUMIDITY DISTRIBUTION UNDER EXISTING C[ONDITIONS]

DISTANCE, 1000's Feet

RELATIVE HUMIDITY AT GROUND LEVEL

74%

75%

76%

77%

78%

DISTANCE, 1000's Feet

HUMIDITY DISTRIBUTION UNDER DESIGN CONDITIONS

71%

72%

73%

74%

5%

DISTANCE, 1000's Feet

DISTANCE IN MILES

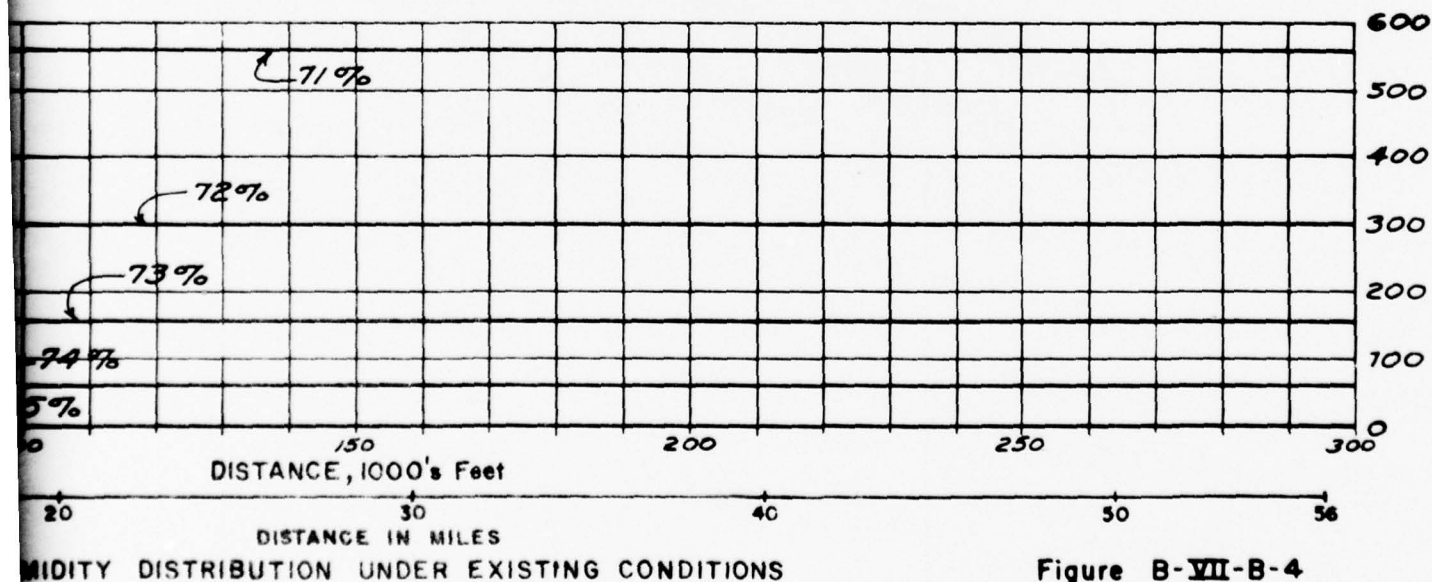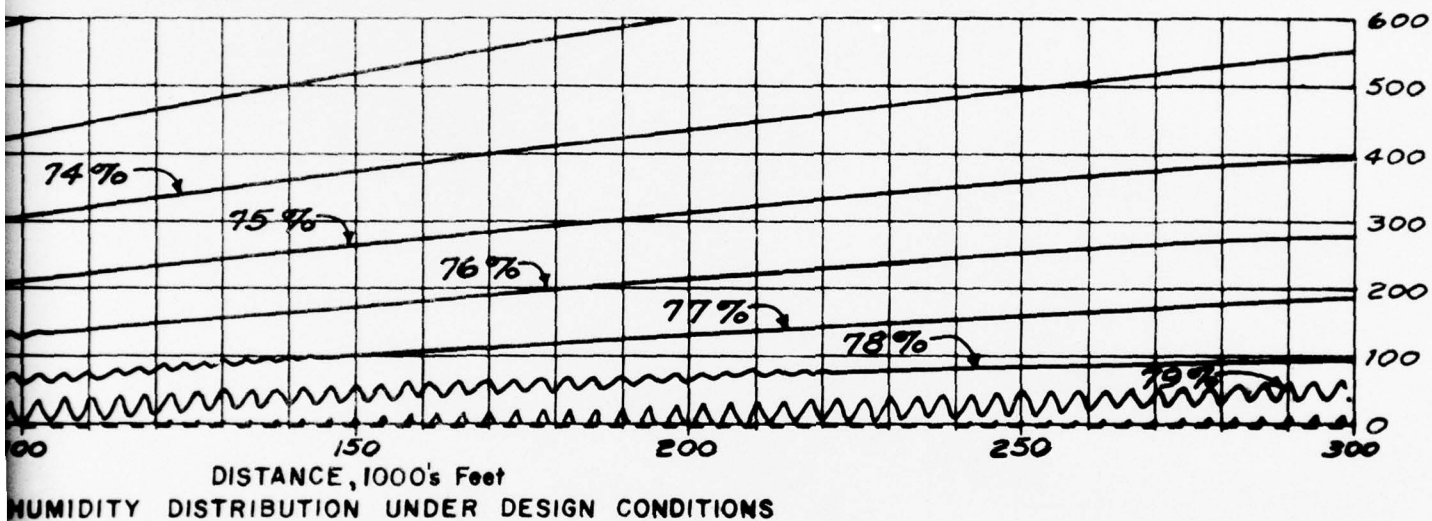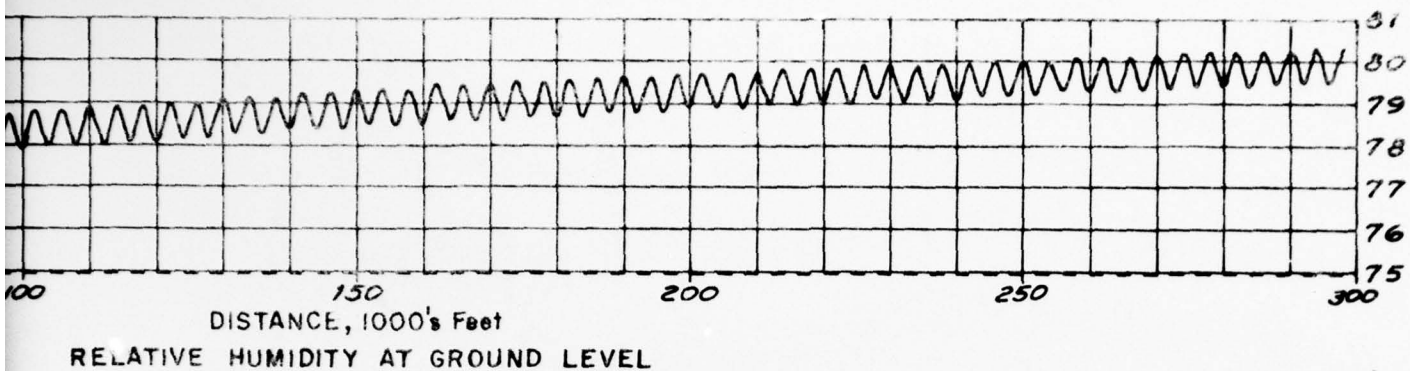HUMIDITY DISTRIBUTION UNDER EXISTING CONDITIONS

Figure B-VII-B-4

PROJECTED HUMIDITY DISTRIBUTION
WITHIN A LAND TREATMENT
IRRIGATION AREA

B-VII-B-21

ing winds for 50 miles. The humidity distribution pattern does not change significantly from the distribution pattern at points beyond 50 miles.

A relative humidity analysis is also made for different wind velocity conditions and for different background humidities. The results of this analysis indicates that wind velocity has no significant effect on the ambient humidity in the irrigation area. Even though more moisture is lost to the atmosphere during period of high wind velocity, these stronger winds have a better capability for diffusing moist air into the atmosphere and this effect counter balances the additional moisture uptake preventing any significant change in the ambient humidity. The effects of irrigation on relative humidity are more significant when the background humidity is low than when the background humidity is high.

The impact that the land treatment irrigation system has on the amount of rainfall in the irrigation area is insignificant in light of the evaluation done for this study. The potential evaporation, represented as lake evaporation, for the proposed irrigation area has the monthly rate distribution shown in Figure B-VII-B-5. Remembering that the evapotranspiration rate depends upon the availability of water in the soil, the evapotranspiration rate for the area without irrigation for an average year is estimated to be 60% of lake evaporation. Assuming that the rate of evapotranspiration equals the rate of lake evaporation during the irrigation season (from April 1 to Nov 31) for the irrigation lands, the increase in annual evapotranspiration loss is estimated to be nine inches over an area of approximately 550 square miles. Since rainfall occurring in one area is frequently evaporation from another area a few hundred miles away, the increase in the precipitation in the vicinity of the irrigation area is estimated to be less than one inch.

GROUNDWATER AT LAND TREATMENT SITES

Present Conditions

In most of the rural areas considered as possible land treatment irrigation sites, the present groundwater drainage is accomplished using only two basic components, a simple man-made drainage system utilizing drain tiles and the natural drainage system of the area.

Although parts of the proposed land treatment irrigation areas already have drain tile drainage systems, these systems are installed for the relief of groundwater flooding in the crop root zone immediately
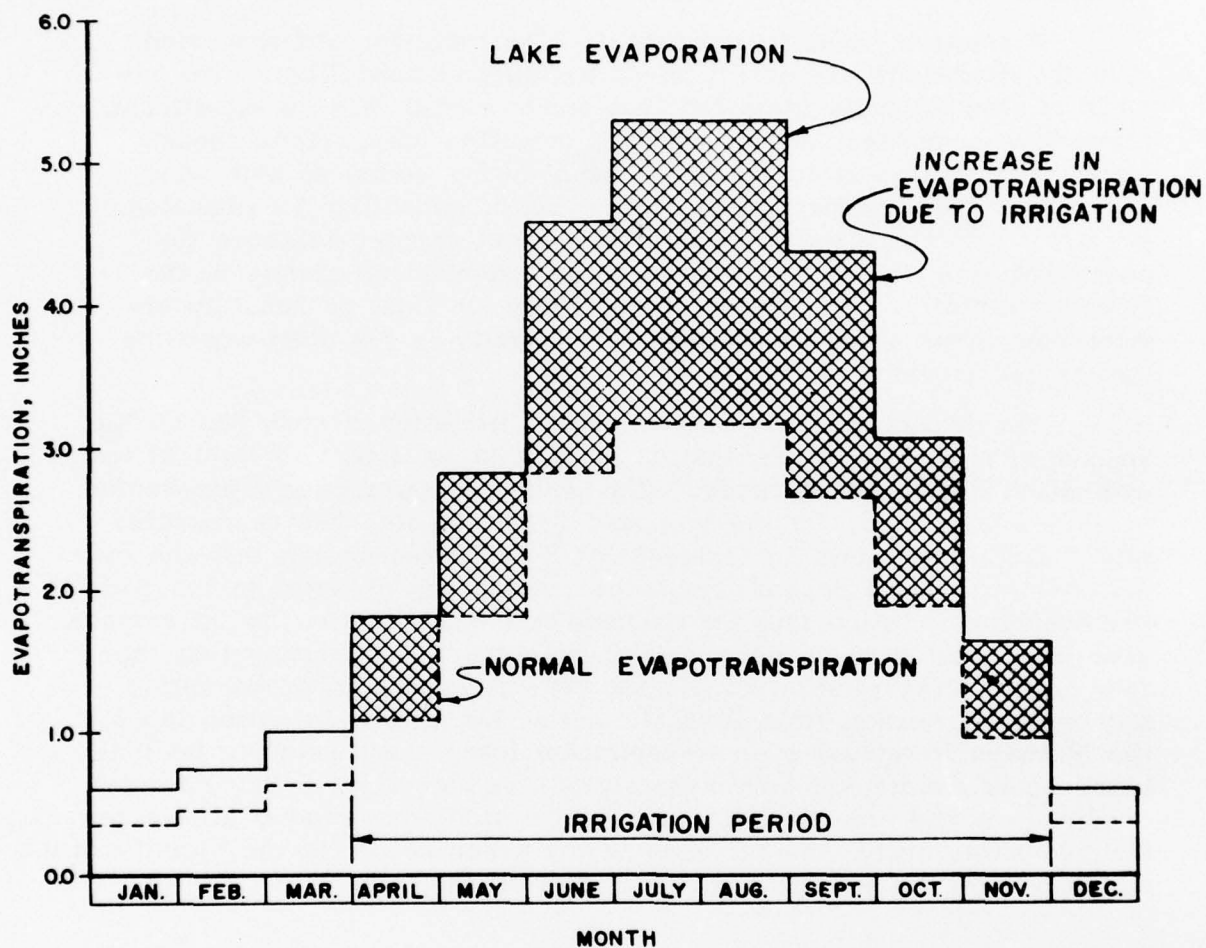
Figure B-VII-B-5

THE EFFECT OF IRRIGATION ON
PRESENT EVAPOTRANSPIRATION PATTERNS

after a heavy storm. When the groundwater table is lower than this installed drain tile system, all of the groundwater is drained through the natural drainage system and eventually enters a nearby creek or ditch. Thus, in the present system, the groundwater table fluctuates from time to time depending on the amount of water supplied to the area.

Land Treatment System

In the proposed land treatment irrigation sites, the groundwater table is maintained at an almost constant level during the irrigation season. To do this the drainage system is designed so that the large amounts of wastewater applied through irrigation are collected in a specially-designed system of drain tiles and pipe. This drain tile system collects the water at a uniform rate according to the irrigation flow provided, and thus maintains almost-constant groundwater levels at the irrigation sites. Details of this drainage system, including the variations in the groundwater table during the irrigation season, are discussed in Appendix B, Section IV-A.

The impacts of the proposed irrigation system on the rural groundwater can be divided into two aspects—the effects it has on the groundwater table and the effects it has on groundwater quality. These two subjects have been investigated for this study with the following results.

Effects on groundwater table. The proposed land treatment irrigation system is provided with drain tiles having a minimum depth below ground level of 13 feet, thus maintaining the groundwater table during normal operating conditions at 12 feet below ground level. This system is also designed so that the groundwater table remains at least five feet below the ground surface after the heaviest rainfall on record in the irrigation area. Since the root zone of most crops extends to some three feet below ground, this system prevents any crop damage that might occur due to high groundwater conditions.

At present, in the proposed land treatment irrigation areas, the existing drainage systems are at an average depth of four feet.[9] For the hydrologic conditions which occurred in 1954, it is estimated that the groundwater table would remain in the crop root zone for more than four days with the present drainage system, thus causing possible crop damage. The 1954 hydrologic conditions are projected to occur, from a probability analysis, every five years. On the other hand,

under the proposed irrigation and drainage system, it is estimated that crop damage would not occur during the 100-year-storm hydrologic conditions. This improvement results from the increased depth and capacity of the drain tile which provides sufficient storage and drainage capacity for excessive amount of stormwater.

Effects on groundwater quality. The reclaimed water from the land treatment system contains a higher concentration of dissolved solids than is contained in the agricultural area groundwater, but is still of potable quality. Nevertheless, in order to maintain isolation between the agricultural area water supply and the C-SELM reclaimed water, specific design considerations have been introduced into the agricultural area drainage system. These design considerations include a variety of techniques for groundwater management and are tailored to the topographical and geological characteristics encountered in the agricultural areas. A description of some of the applicable techniques is contained in the following paragraphs. In an actual detailed design for an agricultural area, these and other drainage techniques would be employed to maintain groundwater and reclaimed water isolation.

The typical geological formation in the McHenry County, Illinois area, where silt loam soils are located, is covered by a glacial drift with a thickness of about 200 ft. This glacial drift contains thick deposits of sand and gravel in two zones: one near the surface comprising an upper aquifer; and the second, immediately above the bedrock comprising the lower aquifer. The upper and lower aquifers are separated by clay-type materials, which form a reasonably impervious barrier to groundwater flow. Wells which pump water from lower aquifers would therefore, be recharged from a far enough area so that either the time required for the reclaimed water to reach these wells is considerably long (several thousand years) or the aquifer would be completely recharged from a nonirrigated area. Thus, a potable supply from this lower aquifer in a McHenry agricultural area would continue to have a water quality (including dissolved solids content) identical with that currently available from this aquifer after installation of a land treatment irrigation system.

The sandy soil type model is used for areas covered with a glacial drift of more than 50-feet in thickness as commonly occurs in the Kankakee drainage basin. The bedrock immediately beneath this

glacial drift is either dolomite of the Silurian or an impervious shale of the Devonian Age. The glacial drift contains thick deposits of sand and gravel which have high permeabilities. For those Kankakee areas where impervious shale of Devonian Age lies in between the glacial drift and the dolomite of silurian age (thus constituting an aquiclude), the groundwater management analysis is similar to the analysis described for McHenry County. The groundwater quality would therefore be maintained at the level presently existing in the dolomite aquifer. For the Kankakee, with glacial drift directly over-lying the silurian dolomite, the reclaimed water from the irrigation area requires tailored management design considerations. The following discussion on these design considerations and related groundwater movement is restricted to the area without the Devonian formation.

In areas with low or mild slopes, the flow pattern of the groundwater inside and outside the irrigation area during the early to middle (normal) irrigation season will be as shown in Figure B-VII-B-6. The groundwater flow pattern demonstrates that there will be no ground-water movement from inside the irrigation area toward the outside of the irrigation area except during the storage season. Because of the high transmissibility associated with the Kankakee glacial drift, the differ-ence between the groundwater levels at a well location and a drain tile would be on the order of few inches during the normal irrigation season. When the storage season arrives during the latter part of the irrigation season, the groundwater table in the irrigation area will build up to a level approximately six feet above drain tile level. There will, therefore, be a movement of groundwater from the irrigation area towards the well area during the storage season. A reverse flow will occur away from the well area and towards the irrigation area during the drainage season which comes after the termination of the irrigation season. In order to clarify the effect of this reversible and oscillatory flow, a hydraulic analysis has been conducted for this system. The results of this analysis are presented in Figure B-VII-B-7. The results of the analysis show that there will be an amount of water flowing out of the well area and into the irrigation area during the winter drainage season approximately equal to that which will be flow-ing from the irrigation area into the well area during the storage season. The effect of rainfall, which has been ignored in the above analysis, tends to increase the reverse flow, thus further preventing the in-trusion of reclaimed water into the well area. As a result, no re-claimed water from the irrigation area would flow into the well or to-wards the well area at any time as a result of a complete year cycle.
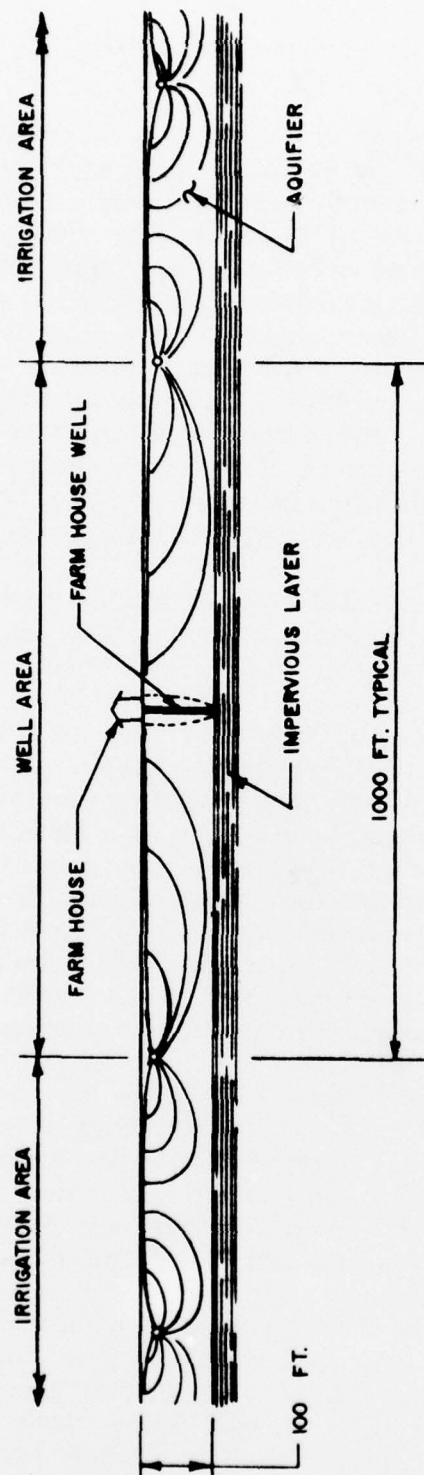
Figure B-VII-B-6
SCHEMATIC FLOW PATTERN OF GROUNDWATER
DURING NORMAL IRRIGATION SEASON
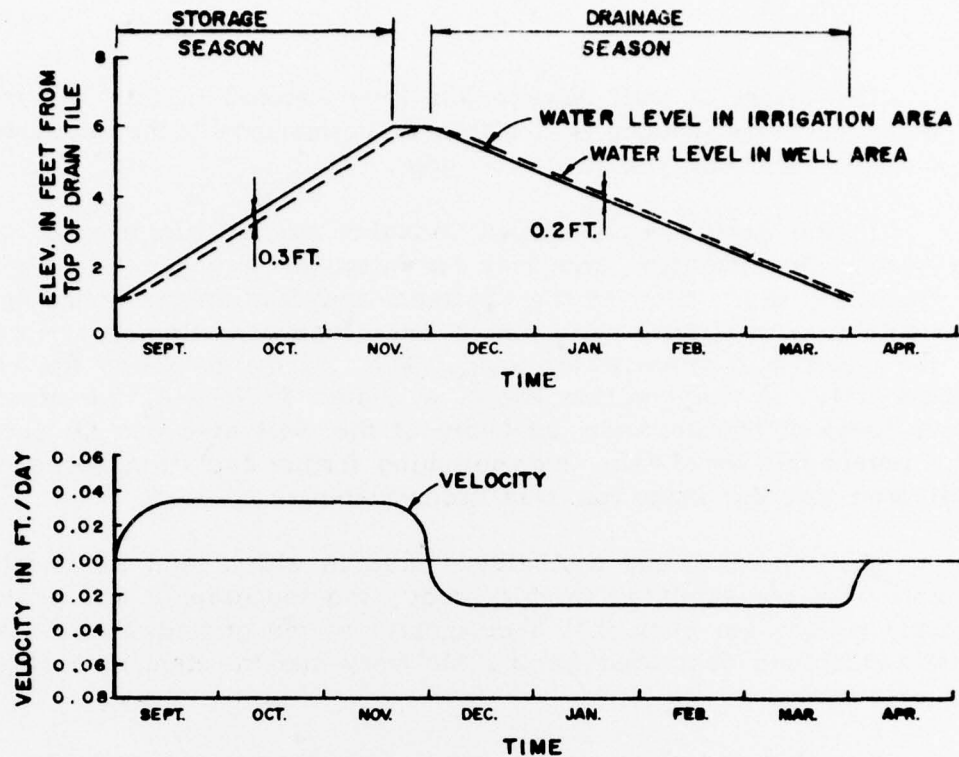
NOTE : THE BASIC EQUATION USED IN THE ANALYSIS IS

$$\frac{d(H-h)}{dt} = \frac{-Km}{fS^2}(H-h)$$

WHERE :   H = WATER LEVEL IN THE IRRIGATION AREA

  h = WATER LEVEL IN THE WELL AREA

  K = PERMEABILITY

  m = AQUIFER THICKNESS

  f = STORAGE COEFF. OF THE AQUIFER

  S = DISTANCE BETWEEN BOUNDARY OF IRRIGATION AREA AND THE FARM HOUSE WELL.

Figure B - VII - B - 7

RESULTS OF THE GROUNDWATER ANALYSIS

The effect of well damage has been ignored in this analysis because this flow amount is neglible in comparison to the oscillatory flow associated with the irrigation area.

In the more steeply-sloped Kankakee areas, migration of ground-water from the irrigation area may be mitigated or completely prevented by installing drain tiles at the upstream and downstream boundary of the well area at depths such that a low slope condition is maintained on the underlying groundwater level. Further, by installing the storage control valve at the position shown in Figure B-VII-B-8, the ground-water level at the upstream boundary of the well area can be suppressed to a reasonable level thus accomplishing further isolation between the well area and the irrigation area groundwater.

For the remaining agricultural area in which land treatment by irrigation is contemplated in this study, the topographic and geologic characteristics are such that a composite of the groundwater management techniques described for the McHenry and the Kankakee areas are applicable.

The detailed analysis for these and other tailored drainage design considerations would be conducted in the detailed design phase for an irrigation system. For the purposes of this survey-scope study, the cost necessary for insuring groundwater isolation is the appropriate cost for extending agricultural area well depths into the lower glacial drift aquifer or the silurian dolomite aquifer, whichever is applicable.
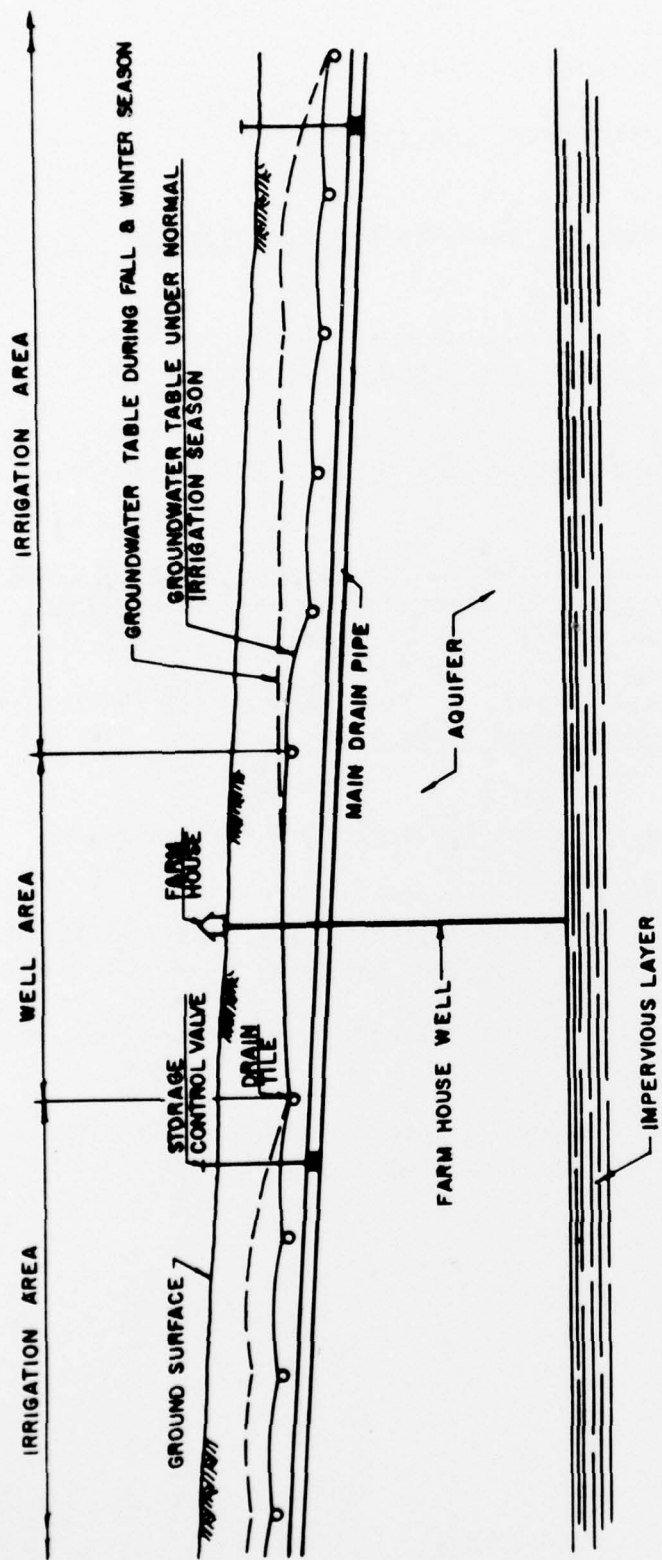
Figure B-VII-B-8
MODIFIED DRAINAGE SYSTEM

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VII.   IMPACTS OF MANAGEMENT SYSTEMS

C.   SYSTEM PERFORMANCE

RELIABILITY OF WASTEWATER MANAGEMENT SYSTEMS

The performance of wastewater management systems can be exa-
mined for the varying operating conditions which the system may en-
counter.   The reliabilities of the mechanical components comprising the
management systems are satisfactory under varying flow conditions be-
cause sufficient maintenance and backup facilities have been designed
into these facilities.   These include backup pumping facilities and
power generation equipment for all of the management conveyance sys-
tems be they pumping wastewater, stormwater, sludge or reclaimed wa-
ter.   The treatment facilities also have backup facilities for critical
treatment components such as lift station pumps, blowers, return acti-
vated sludge pumps, sludge pumps, digestion facilities, chemical feed
systems, lime recalcination and carbon regeneration facilities, activated
carbon wastewater pumps, ion exchange regenerant pumps and air blowers,
filtration backwash pumps, surface mechanical aerators and irrigation
pump station facilities.   On the other hand, the reliabilities of the treat-
ment plant biological components can be compromised by varying influent
wastewater characteristics.   For example, accidental industrial toxic
spills to the sewer system may kill the treatment systems' biological
community thereby decreasing the performance and reliability of the sys-
tem.   The biological component of the land treatment system is intrisi-
cally capable of greater reliability under these circumstances.   The
much increased retention time in the land treatment biological system
dilutes the effect of wastewater quality fluctuations to a more manage-
able level for biological systems.   Furthermore, the extremely large
storage and dilution provided by the land treatment storage lagoon pro-
vides a final guarantee of biological performance reliability.

During wet weather conditions, conventional secondary plants
exhibit poor effluent quality due to the overtaxing of the treatment units
caused by peak infiltrated stormwater flows.   For the AWT management
systems, the reliability of the system to conform to the NDCP effluent
standard during these wet weather conditions is very good.   This is
due to stormwater management systems providing proper stormwater run-
off storage facilities so as to prevent or minimize the pollutional im-
pact of very large storms on the receiving streams in the C-SELM area.

In a like manner, the land treatment drainage system is designed to adequately handle the maximum storm on record and thus prevent extensive wet land conditions and resultant crop losses. However, the land system is susceptible to extreme weather conditions. System performance may be locally impaired by high winds, such as encountered during tornadoes that are experienced in the midwest, with resultant damage to local irrigation facilities and with local agricultural crop losses. The storage facilities of the land system should provide sufficient detention time to correct such system damage and thus minimize the performance impact from these natural weather disasters.

The AWT systems' reliabilities are suspect when considerations are given to the long-term availability of resources required by these systems. For example, the treatment plant systems are designed to utilize large quantities of natural gas which is presently in large demand in the midwest. Since present priorities are given to low-level consumers such as homes and commercial buildings, it is possible that large treatment facilities would be considered a low priority in the issuance of natural gas supplies. Alternative fuel sources such as gas produced from coal gasification processes may provide the answer for the insurance of future system reliability. However, this process is in the research and development stage and is not yet ready for large scale purposes. Alternative fuels, such as coal and oil, cannot be considered without proper management of the increased pollutant loadings to the air resource, mainly in terms of sulfur dioxide emissions.

The reliability of the physical-chemical system to control nitrogen oxide emissions from the fluidized-bed reactors is very questionable in light of the present state of the art for controlling nitrogen oxide emissions. Again, future technological advances such as the refinement and large-scale development of a nitrogen oxide catalytic reduction process may solve this problem. However, at present, it can be stated that the physical-chemical nitrogen control system for the air resource cannot be relied upon to meet existing air quality standards. The nitrogen control process for the advanced biological system also presents resource reliability considerations. Methanol, which is designed into the denitrification process to provide a carbon source for the bacteria, is manufactured from natural gas. There are, however, a number of more costly alternative carbon sources to replace methanol. Thus the major impact on this system performance consideration would be one of economics.

Presented in Table B-VII-C-1 are the various types of treatment or conveyance units associated with the wastewater management systems. Associated with each unit is a reliability factor which is rated on a scale from 0 to 1 with 0 being the least reliable and 1 being the most reliable. The determination of the reliability of a wastewater management system is, at best, a rather subjective undertaking. However, it is felt that with a comprehensive understanding of these systems, the reliability factors for the varied number of management components may facilitate a determination of the relative complexity of the AWT systems.

ADAPTABILITY OF WASTEWATER MANAGEMENT SYSTEMS

New technological advances may replace the need for particular wastewater management system components or unit processes; the adaptability of these units for other uses is, therefore, a consideration in the overall performance of the system. For example, such facilities as mixers, aerators, chlorinators, lime slakers and sludge disposal tractors could be readily adapted to other uses in water treatment or industrial and agricultural applications.

Unless a treatment technology is developed for use in the home, the wastewater collection and conveyance systems will always be necessary for transmitting the wastes to treatment facilities regardless of the treatment technology involved. On the other hand, if a treatment plant technology replaces the land treatment system, the land conveyance wastewater tunnel system might not be adaptable for other uses. Again if the land system is replaced, the irrigation and drainage systems are still quite adaptable for high crop yield agricultural programs in these rural areas. The storage and aerated lagoon facilities also may have recreational development values in these rural areas.

For the treatment plant systems, little adaptability can be assumed for ion exchange or carbon adsorption columns. Also, the large concrete tank structures would provide little adaptability unless they were used for stormwater storage facilities or the like. Air blowers from the aeration and fluidized-bed facilities may be adaptable to certain industrial applications as would the variety of chemical feed systems. Carbon regeneration and lime recalcination facilities may also be adaptable for other incineration purposes, namely municipal refuse applications.

Also presented in Table B-VII-C-1 are adaptability factors for the various wastewater management system components. These factors are rated on a scale from 0 to 1 with 0 being the least adaptable and

MEASURE OF COMPLEXITY

(Values for treatment systems represent the number of units requir

Capital Units	Adaptability Factor	Reliability Factor	Conventional Biological Treatment		Physical-Ch Treatme	
			1990	2020	1990	2
Grit Tanks	0.4	0.9	4.0	4.0	4.0	
Mixers	0.9	0.8	-	-	2.0	
Clarifiers	0.4	0.8	32.0	32.0	8.0	
Thickeners	0.3	0.8	4.0	2.0	6.0	
Vacuum Filters	0.5	0.5	-	-	8.0	
Air Classifiers	0.4	0.5	-	-	2.0	
Incinerators	0.5	0.3	-	-	6.0	
Slakers	0.5	0.5	-	-	5.0	
CO_2 Compressors	0.6	0.7	-	-	4.0	
Adsorption Columns	0.3	0.6	-	-	53.0	5
Ion Exchange Columns	0.3	0.6	-	-	52.0	5
Filter Beds	0.4	0.7	-	-	14.0	1
Chlorinators	0.7	0.7	2.0	2.0	2.0	
Aeration Tanks	0.4	0.9	12.0	12.0	4.0	
Blowers	0.6	0.8	3.0	3.0	-	
Digesters	0.3	0.3	8.0	8.0	-	
Denitrification Tanks	0.4	0.4	-	-	-	
Aerators	0.7	0.8	-	-	-	
Irrigation Rigs	0.7	0.7	-	-	29.0	1
Lagoon Berm Footage	0.6	0.9	-	-	-	
Irrigation Pressure Pipe (miles)	0.6	0.9	-	-	5.1	
Drainage Pipe (miles	0.6	0.9	-	-	50.0	3
Conveyance and Collection Conduits (miles)	0.1-0.9	0.9	36.2	31.0	168.0	16
Pumping Stations	0.1-0.9	0.9	6.1	5.1	19.1	1
Tractor - Plow or Spray	0.8	0.7	0.9	0.9	-	
Tractor - Sprinkler	0.8	0.7	0.6	0.6	38.2	3
Chemical Feeders	0.5	0.6	0.1	0.1	-	
Dredges	0.3	0.5	-	-	-	

Table B-VII-C-1

URE OF COMPLEXITY

nt the number of units required for a 100 MGD capacity system)

Conventional gical Treatment		Physical-Chemical Treatment		Advanced Biological Treatment		Land Treatment	
0	2020	1990	2020	1990	2020	1990	2020
	4.0	4.0	4.0	4.0	4.0	4.0	4.0
	–	2.0	2.0	2.0	2.0	34.0	34.0
	32.0	8.0	8.0	64.0	77.0	–	–
	2.0	6.0	6.0	10.0	12.0	–	–
	–	8.0	8.0	8.0	9.6	–	–
	–	2.0	2.0	2.0	2.0	–	–
	–	6.0	6.0	6.0	6.0	–	–
	–	5.0	5.0	5.0	5.0	–	–
	–	4.0	4.0	4.0	4.0	–	–
	–	53.0	53.0	53.0	53.0	–	–
	–	52.0	52.0	–	–	–	–
	–	14.0	14.0	4.0	14.0	–	–
	2.0	2.0	2.0	2.0	2.0	2.0	2.0
	12.0	4.0	4.0	22.0	26.4	–	–
	3.0	–	–	6.0	7.2	–	–
	8.0	–	–	8.0	9.6	–	–
	–	–	–	2.0	2.0	–	–
	–	–	–	–	–	–	–
	–	29.0	17.0	29.0	17.0	162.0	151.0
	–	–	–	–	–	43.4×10^3	41.7×10^3
	–	5.1	3.2	5.1	3.2	70.0	66.8
	–	50.0	30.6	50.0	30.6	96.8	94.6
	31.0	168.0	160.0	50.0	43.5	59.1	51.3
	5.1	19.1	14.1	14.7	9.9	14.8	10.1
	0.9	–	–	3.5	3.5	3.3	3.3
	0.6	38.2	37.5	0.9	1.0	0.8	0.7
	0.1	–	–	0.1	0.1	0.1	0.1
	–	–	–	–	–	0.2	0.2

1 being the most adaptable. These adaptability factors together with the number of units involved may provide some insight into how recoverable or adaptable the system is for some purpose other than to treat wastewater.

DISRUPTION

Implementation of any of the wastewater management alternatives described in Appendix D will have some degree of disruptive effect on the life styles of the C-SELM area. The resulting relative disruption can be predicted by a variety of means among which are: examination of selected impact considerations including impact on land displacements and people displacements. These impacts, by alternative wastewater management technologies, are discussed in Appendix B, Section VII-B. The greater the people and land displacements, the greater can be the anticipated C-SELM disruption.

As presented in Table B-VII-C-1, the cumulative flow conduit lengths of all the combined conduit sizes associated with an AWT technology are summed and displayed as an entry in the measure of complexity tabulation. Surface disruption is a significant function of the amount of conduit required to implement the wastewater management systems. The more regional the treatment approach, the greater the collection and conveyance disruption in the C-SELM area. However, the construction of wastewater conveyance tunnels will cause less disruption than the construction of near-surface sewers or force mains.